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Volume 29

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In this issue: A study of nighttime legibility of highway signs with white reflectorized letters on a dark background (jct. of U. S. 1 and Virginia Route 350, Arlington County).

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Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretary

BUREAU OF PUBLIC ROADS
CHARLES D. CURTISS, Commissioner

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The Effect of Letter Width and Spacing on Night Legibility of Highway Signs

*BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS*

**Reported [1] by DAVID SOLOMON
Highway Transport Research Engineer**

This study was undertaken to determine the effect that spacing between letters of words, used in highway signs, had on their nighttime legibility. More than 2,500 observations were made by 36 observers while driving an automobile at 30 miles per hour. White reflectorized letters, 10 inches high, were displayed on a black nonreflectorized background. Three different alphabets were used. Two of these, the standard Series C with narrow letters and the wider Series E, were cut from reflective sheeting. The third alphabet, identified as Series ED and similar in width to the Series E, was designed by a manufacturer using 1¼-inch-diameter plastic reflectors to form the letters. The spacings between letters were increased as the lengths of the six test words were extended from normal to 20, 40, and 60 percent above normal.

As interletter spacing was increased, the legibility distances also increased for all three alphabets until word lengths were 40 percent above normal. The resulting gain in legibility at this point was 15 percent for Series C, 16 percent for Series E, and 7 percent for Series ED. Beyond the 40-percent increase in word length, legibility leveled off or declined.

When word lengths were normal or no more than 10 percent above normal, test signs with the Series ED alphabet were found to have greater legibility. At wider spacings, the Series E alphabet was superior.

As might be expected, the 10-inch Series E alphabet was legible at a greater distance (118 to 142 feet) than the narrower 10-inch Series C alphabet at corresponding letter spacings. On a percentage basis, the differences in legibility favoring the Series E alphabet ranged from 23 percent to 27 percent. A word with letters of the Series C alphabet is shorter in length than one with letters of the Series E for a given spacing, and a comparison of legibility distance per inch of word length showed that the Series C alphabet was somewhat superior to the Series E. Also studied was the probable effect of increasing the letter height of the narrower alphabet until the legend area equaled that of the wider alphabet. At the point of equivalent legend area and spacing, the two alphabets proved to be equally legible.

The study findings point to the importance of sign proportions and provide an improved means for efficient determination of legend design. Where vertical dimensions restrict sign letter heights to something less than desirable, increased spacing between letters can help to compensate for the loss of legibility distance that would otherwise occur.

IN recent years, increasing use has been made of white reflectorized letters on a dark background for large highway destination signs. Although there are some conflicting reports, the weight of the evidence indicates that, for night legibility of large letters, this combination is superior to dark letters on a white reflectorized background. Consequently, a design using white reflectorized letters on a dark background was selected for signs recently installed on the Pentagon road network just outside of Washington, D. C. Standard alphabet designs for highway signs developed by the Bureau of Public Roads were employed.

The letter spacings used for the design of the Pentagon network signs were originally devised from limited tests and previous researches dealing with letter spacing and legibility, all with black letters on a white background. The spacing values were selected so that the white areas between successive letters in a given word would appear equal.

It was surmised that if the letter spacings were not changed the color and reflectorization reversal on the new signs might have a significant effect on night legibility. Furthermore, a recent laboratory study [2] had shown that white reflectorized letters on a black background may improve in legibility if the letter spacing is somewhat wider than that used with black on white. No definitive research looking toward the establishment of optimum spacings between letters had been conducted under actual roadway driving conditions.

The present study, therefore, was initiated to investigate the effect on nighttime legibility of increasing the spacing between white reflectorized letters on a black background.

Study Conditions

The study site was a bituminous-surfaced parking lot, 1,500 feet in length, substantially level and unused at night. The area was without illumination except for a few fixed street lights in the distant background. Thus the conditions were similar to those found on unlighted urban freeways or rural highways near cities.

Studies were conducted during March and April of 1955. Observations began after dark and continued until 11 p. m. or midnight. Generally, the weather was fair and no moon was present, although some data for 4 of the 36 observers were recorded during a light rain, and the observations of 2 others were recorded in the presence of a quarter moon. The light rain and moonlight had no apparent effect on the results obtained.

Observers were 36 male employees of the Bureau of Public Roads and of the District of Columbia Department of Vehicles and Traffic. They ranged in age from the middle twenties to the late sixties, and 20 wore eyeglasses. The participants were chosen largely because of their willingness to work in the evening hours, rather than for any particular physical ability or characteristic.

All but 2 of the 36 observers drove one of three 1951 model Pontiacs. A fourth Pontiac, 4 years older, was used by the remaining observers. The test vehicles were equipped with the improved "50–40" sealed-beam headlamps to simulate the best possible visual conditions. The headlamps were checked periodically for proper aim. The new "50–40" headlamps are now legal in all States and all vehicles in production are being equipped with them.

A test panel, 6 feet long by 4 feet high, was mounted at one end of the parking lot with the lower edge 4½ feet above the pavement. Space

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[1] This article was presented at the 35th Annual Meeting of the Highway Research Board, Wash., D. C., Jan. 1956.

[2] *Analysis of certain variables related to sign legibility* by H. W. Case, J. L. Michael, G. E. Mount, and R. Brenner. Highway Research Board, Bulletin 60, 1952, pp. 44–58.

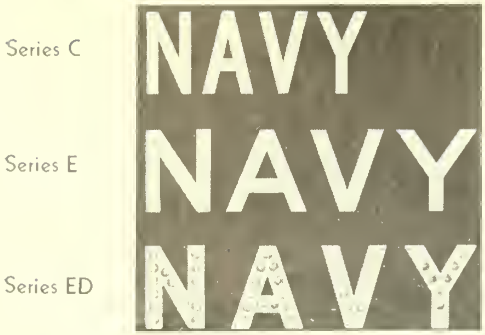

Figure 1.—Comparison of Series C, E, and ED alphabets at normal spacing.

Table 1.—Character (letter or numeral) width and normal spacing of 10-inch Series C and E standard alphabets and 10-inch Series ED (manufacturer's alphabet)

Character	Standard alphabets				Manufacturer's alphabet		
	Character width		Edge code [2]		Character width of series ED [3]	Margin spacing	
	Series C [1]	Series E [1]	Left	Right		Left	Right
	Inches	*Inches*			*Inches*	*Inches*	*Inches*
A	6.25	10.00	III	III	9.00	1.1	1.2
B	5.47	7.97	I	II	7.38	2.2	1.5
C	5.47	7.97	II	III	7.38	1.2	1.4
D	5.47	7.97	I	II	7.38	2.2	1.3
E	5.00	7.34	I	III	6.75	2.2	1.4
F	5.00	7.34	I	III	6.75	2.2	1.0
G	5.47	7.97	II	II	7.38	1.2	1.4
H	5.47	7.97	I	I	7.38	2.2	2.3
I	1.41	1.72	I	I	2.12	2.2	2.3
J	5.00	7.50	III	I	6.75	.8	2.3
K	5.47	8.12	I	III	7.62	2.2	.9
L	5.00	7.34	I	III	6.75	2.2	.9
M	6.48	9.22	I	I	8.50	2.2	2.3
N	5.47	7.97	I	I	7.38	2.2	2.3
O	5.78	8.28	II	II	7.62	1.2	1.3
P	5.47	7.97	I	II	7.38	2.2	1.3
Q	5.78	8.28	II	II	7.62	1.2	1.3
R	5.47	7.97	I	II	7.38	2.2	1.3
S	5.47	7.97	II	II	7.38	1.2	1.1
T	5.00	7.34	III	III	6.75	.9	1.0
U	5.47	7.97	I	I	7.38	2.2	2.3
V	6.09	9.06	III	III	8.12	1.1	1.2
W	7.50	10.47	III	III	9.38	1.1	1.2
X	5.86	8.59	III	III	7.62	1.1	1.2
Y	6.25	10.00	III	III	9.00	1.0	1.1
Z	5.47	7.97	III	III	7.38	2.0	2.1
1	2.03	2.97	I	I	3.12	2.0	2.3
2	5.47	7.97	II	II	7.38	1.2	1.1
3	5.47	7.97	III	II	7.38	1.3	1.5
4	6.09	9.22	III	III	8.25	1.2	2.0
5	5.47	7.97	I	II	7.38	1.1	1.2
6	5.47	7.97	II	II	7.38	1.3	1.2
7	5.47	7.97	III	III	7.38	.9	1.0
8	5.47	7.97	II	II	7.38	1.2	1.3
9	5.47	7.97	II	II	7.38	1.2	1.3
0	5.78	8.28	II	II	7.62	1.2	1.3

[1] Stroke width for Series C, 1.41 inches; Series E, 1.72 inches.

[2] Edge codes determined, with minor exceptions, as follows: Code I indicates side of character has vertical outline; code II, curved outline; and code III, diagonal or open-faced outline. For measurements and sample calculations, see tables 2-3.

[3] Stroke width for Series ED, 2.12 inches. Reflectors measure 1¼ inches in diameter.

was provided for two 4-letter words placed one above the other with a 10-inch clearance between lines. Individual letters 10 inches in height were combined to form the test words. Because individual letters were used on the panel, the spacing between letters could easily be varied to suit the requirements of each observation. Narrow guide strips were placed below the words to indicate the letter position for each test condition. The guide strips were not reflectorized or visible during the test runs.

Scope of Study

Three different alphabet designs were observed. Figure 1 shows the word NAVY displayed in each alphabet.

Series C and Series E alphabets, 10 inches in height, were chosen because they are representative of those commonly appearing on highway destination signs. These two alphabets were cut from reflective sheeting. The sheeting employed reflects nearly as much light at 20- or 30-degree angles of incidence as it does head on.

In a third alphabet, identified in this study as Series ED, 1¼-inch-diameter plastic reflectors formed the 10-inch high letters. The brightness of these units was considerably greater than the sheeting used for the Series C and E alphabets. The Series E alphabet was quite similar in form to the Series ED, which was designed by a manufacturer.

All three alphabets were made from materials with retrodirective properties; i. e., a tendency to reflect a large portion of the light back to the source regardless of the angle of the incident light. The sheeting from which Series C and E alphabets were cut is composed of minute glass spheres, while the plastic units used for the Series ED alphabet develop their reflection from the interior corners of cubes.

From each of the three alphabets, six test words were composed—BALK, FARM, NAVY, STOP, ZONE, and DUCK. These six words use 19 of 26 letters in the alphabet and have a variety of letter forms adjoining each other.

Each of the six words was displayed in three alphabet designs and four different spacings for each of the 36 observers. The base or normal spacing was determined by using the Bureau of Public Roads' spacing chart for Series C and E alphabets and the manufacturer's recommendations in the case of Series ED. These values are shown in tables 1 and 2. A sample calculation for determining the length of a word with normal spacing is given in table 3. To increase interletter spacings, word lengths 20, 40, and 60 percent greater than normal were chosen and the individual letters were arranged within that length to produce a satisfactory appearance.

The 72 combinations of alphabets, words, and spacings were displayed on the panel two at a time. The two combinations used for each panel were changed at the halfway point in the test. In this manner, each combination of alphabet, word, and spacing appeared one-half of the time with a word and spacing combination from each of the other two alphabets. The panel displays were systematically chosen so that each word appeared two or three times with every other word. The order of panel presentation was initially selected at random for each half of the test runs, and then the order was advanced for each observer so that each alphabet, word, and spacing combination appeared about the same number of times near the beginning, middle, and end of the test sequences. Also, each combination was shown about the same number of times in the upper position on the panel as in the lower position.

In short, each combination of alphabet, word, and spacing received as favorable treatment as any other, and the 36 observers read the 72 combinations in conformance with a plan designed to minimize every foreseeable bias. This resulted in a total of 2,592 balanced observations.

In addition to the 72 combinations that were displayed two at a time, the 24 combinations of the Series E alphabet were displayed one at a time for 29 of the 36 observers. Only 6 of the 24 combinations were displayed for any single observer, however.

Figure 2 illustrates a partial breakdown of the 2,592 observations. For the 10-inch Series E alphabet, 216 of the 864 observations were made at each of four different word lengths. The same breakdown was used for Series C and ED alphabets. Similar procedures were followed for each succeeding subgrouping of the total observations.

Table 2.—Normal spacing between 10-inch letters of alphabets with combinations of codes I, II, and III for adjacent edges

Examples of letter combinations	Edge code combinations	Letter spacing [1]	
		Series C	Series E
		Inches	*Inches*
HI, JL	I-I	2.11	2.58
NO, PB	I-II or II-I		
NA, FL	I-III or III-I	1.69	2.06
GO, RC	II-II		
ST, ZO	II-III or III-II		
FT, LA, VY	III-III, not parallel.	1.12	1.37
EX, LT, WA	III-III, parallel	.56	.68

[1] Measured horizontally between nearest points.

Table 3.—Sample calculation for determining length of the word NAVY in 10-inch letters of the Series E and ED alphabets with normal spacing

Letter	Width of letters	Edge code		Margin spacing		Combination of codes for adjacent edges	Spacing between letters	Length of word
		Left	Right	Left	Right			
SOLUTION FOR SERIES E								
	Inches			*Inches*	*Inches*		*Inches*	*Inches*
N	7.97	I	I			I-III	2.06	
A	10.00	III	III			III-III, parallel	.68	
V	9.06	III	III			III-III, not parallel	1.37	
Y	10.00	III	III					
Total	37.03						4.11	41.14
SOLUTION FOR SERIES ED								
N	7.38			2.2	2.3		3.4	
A	9.00			1.1	1.2		2.3	
V	8.12			1.1	1.2		2.2	
Y	9.00			1.0	1.1			
Total	33.50						7.9	41.4

Visual acuity for each observer was determined from his test results for all three alphabets and six words at normal spacing. The mean legibility distance for these 18 combinations was the visual acuity score for a given observer. All 36 observers were ranked according to their visual acuity scores and then divided into three visual acuity groups of 12 observers each. This grouping was used in analyzing the effect of visual acuity on legibility.

Conclusions

The following conclusions may be drawn for the nighttime legibility of 10-inch white reflectorized letters on a dark, nonreflectorized background under simulated roadway conditions, and with low beam of the headlamps:

1. Definite improvements in legibility result from moderate increases above the spacings normally used between sign letters. If interletter spacings are increased until the word length is 40 percent greater than normal, the mean legibility distance increases 15 percent above normal for the Series C alphabet, 16 percent for Series E, and 7 percent for Series ED.

2. An increase of approximately 40 percent over the normal word length is about the limit for realizing additional legibility through greater interletter spacing. Words lengthened more than 40 percent above normal gain little or nothing in legibility; with Series E and ED alphabets, word legibility actually declines and the increase with Series C is only slight.

3. Mean legibility distances recorded for the 36 observers were 478 feet for the Series C alphabet, 596 feet for Series E, and 614 feet for Series ED—all measured at normal letter spacing. The 15-percentile values ranged from 27 to 33 percent lower, being 350, 400, and 450 feet for Series C, E, and ED alphabets, respectively.

4. Words formed with the Series ED alphabet have slightly superior legibility in comparison with the Series E alphabet when word lengths are normal or no more than 10 percent in excess of normal. At wider letter spacings, the Series E alphabet is more legible. Letter design details and reflectent characteristics both differed between the Series E and ED alphabets, but an evaluation of the independent contributions of these two factors to legibility was beyond the scope of this study.

5. Legibility distances for words formed with the Series E alphabet are 118 to 142 feet greater than for Series C at the various spacings. The Series C alphabet, however, occupies less word length for a given spacing, and on the basis of legibility distance per inch of word length, it is somewhat superior to Series E. The two alphabets are equally legible when displayed at corresponding spacings and when letters of the Series E alphabet are reduced in height so that the legend areas are the same.

6. A display of two words rather than one on a sign tends to reduce the distance at which any single element of the message is legible. Additional words on the sign would reduce legibility further.

7. The responses of observers grouped according to their visual acuity are similar. Legibility curves determined for these groups,

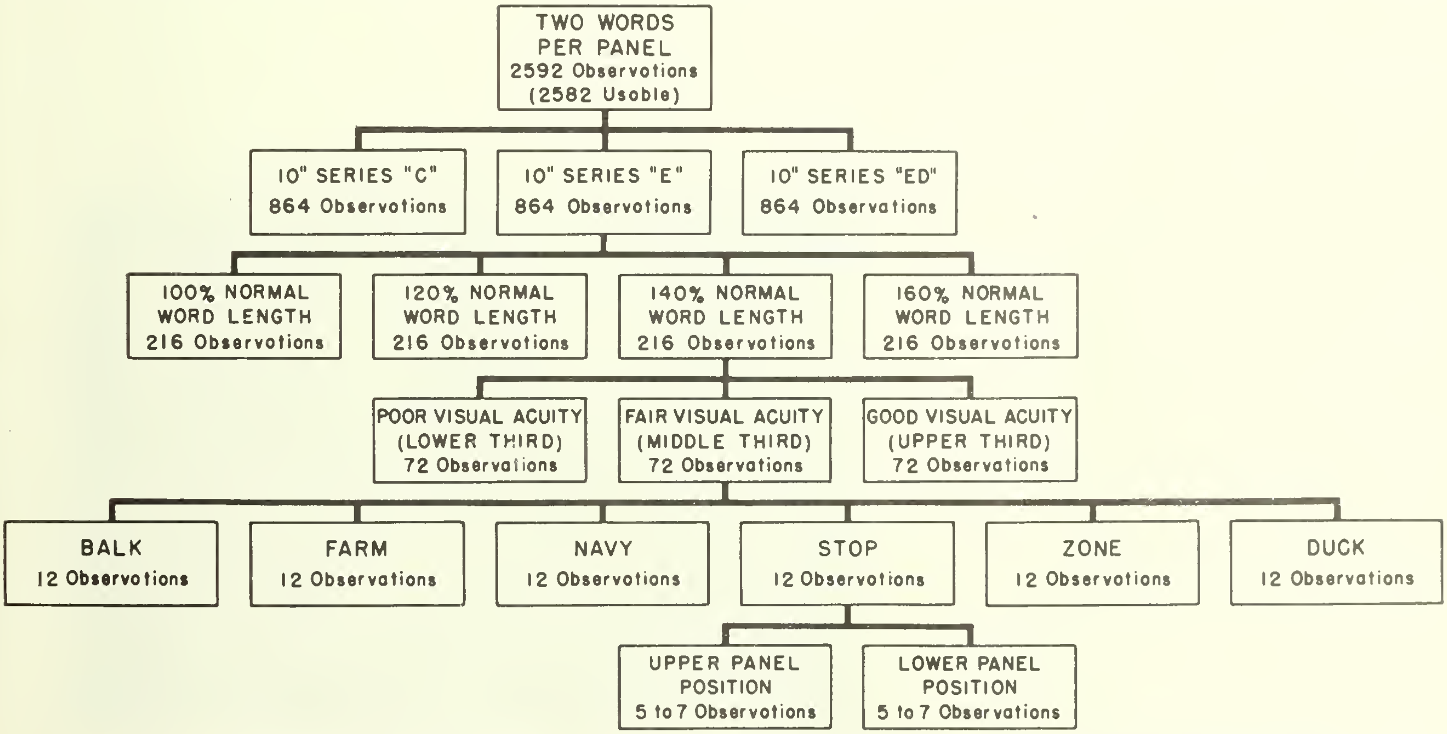

Figure 2.—Number of observations for each alphabet, spacing, visual acuity group, word, and panel position.

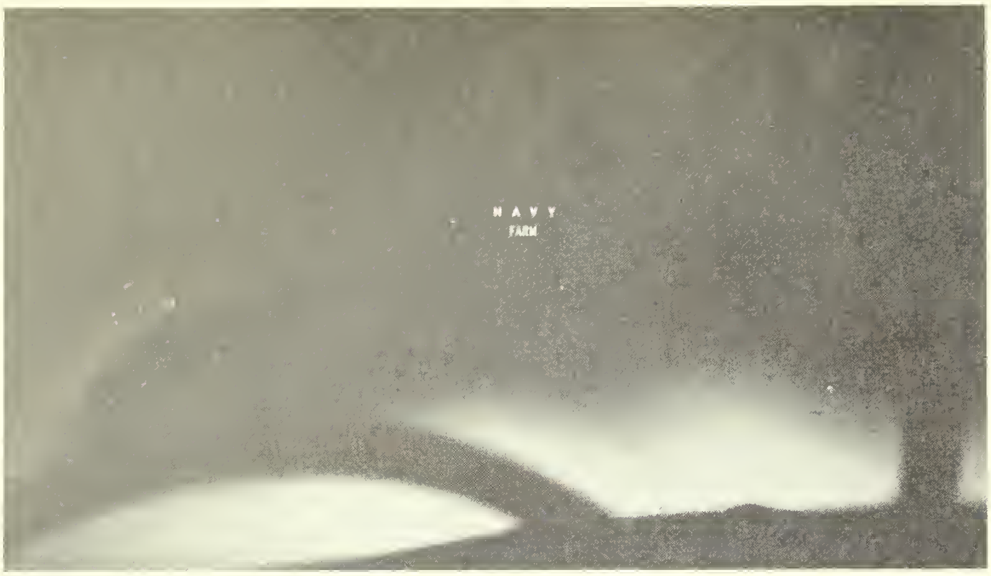

Figure 3.—Appearance of sign panel from a distance of 200 feet.

though of a different magnitude, follow remarkably consistent patterns.

8. A change from 5 to 7 feet in the vertical position of a sign legend above the roadway has little effect on the legibility distance.

9. More consideration of sign proportions is warranted in the development of sign design. Where vertical dimensions are restricted and dictate the use of letter heights less than those desirable, increased word lengths can help to compensate for the loss of legibility distance that would otherwise occur.

10. Within the limitations of appearance and need for emphasis, the horizontal spacings between sign letters should more commonly be increased to take advantage of portions of the sign area otherwise unused.

Test Procedure

Each observer was instructed to drive along the course at an estimated speed of 30 miles per hour, but to focus his attention on the sign panel rather than on the speedometer. He was shown a typewritten list of the six test words before the test began, but was not informed of their order of presentation or about details of alphabet and spacing. Normally, two cars made test runs in close sequence—beginning at one end of the course some 1,500 feet from the sign panel. The first driver was allowed to proceed 600 to 900 feet along the course before the second driver started in order to minimize interference in the legibility observations. The driver made the observations and a recorder sat in the rear seat. No one else was in the car and all runs were made using low beam of the headlamps.

As soon as the driver-observer could read either of the two words, he called it aloud, and the recorder noted the distance from the panel to the nearest 25 feet by referencing his position to coded markings on the course. Similarly, the second word was read as soon as legible and the distance recorded. After both words had been read, the observers continued past the sign panel and drove back to the beginning of the course. Two new words were then placed on the panel. During each circuit of the course, which consumed about $2\frac{1}{2}$ minutes, each observer usually saw the headlamps of the other car although not while actually reading the signs. Thus the effect of light from intermittent opposing headlamps on the eye was introduced to a limited extent, but care was taken to avoid any direct glare.

Observers were instructed to drive so that the left edge of the car was in line with a row of parking stall markings. This resulted in a simulated vehicle placement in the center of a 12-foot lane with the near edge of the sign panel 6 feet from the right shoulder.

The instructions discouraged guessing by observers and few combinations were read incorrectly. At times it was possible to expose the incorrectly read combinations for a second observation, and as a result only 10 observations of the 2,592 were not usable.

Figure 3 shows the words NAVY and FARM from the observer position in a car 200 feet from the sign panel. The stall markings that served as a guide for the observers are dimly visible at the left. Beside the left curb, coded course markers spaced 50 feet apart were used by the recorder to identify the point at which the observer read each combination.

Figure 4 is a closeup view of the panel. NAVY is formed with letters of the Series ED alphabet at the widest spacing (word length, 60 percent greater than normal). FARM is formed using letters of the Series C alphabet at normal spacing.

Legibility Distances Determined

For each of the 36 observers, an average legibility distance was calculated from all 72 observations. These values varied from less than 300 to nearly 900 feet, a ratio of over three to one. Figure 5 shows this relation. Over two-thirds of the observers had average values between 500 and 800 feet, but the distribution was somewhat skewed in the lower direction with one-ninth of the observers having average distances below 400 feet. When highway signs are designed, the capabilities of drivers with poor visual acuity deserve special consideration. Observers wearing glasses did not perform quite as well as a group as those without glasses.

Word lengths increased

For all three alphabets, increasing the spacing first produced greater legibility, then a leveling off or an actual decrease in legibility occurred, as shown by figure 6.

The mean legibility distance for the narrow Series C alphabet at normal spacing was 478 feet. Legibility increased to 549 feet at word lengths 40 percent greater than normal, but the 60-percent increase in word length resulted in only 5 feet more of legibility distance.

The wider Series E alphabet produced a mean legibility distance of 596 feet at normal spacing. A 40-percent increase in word length resulted in a mean legibility distance of 691 feet, while the 60-percent increase in word length actually reduced the legibility distance by 10 feet.

For the Series ED alphabet, similar in form

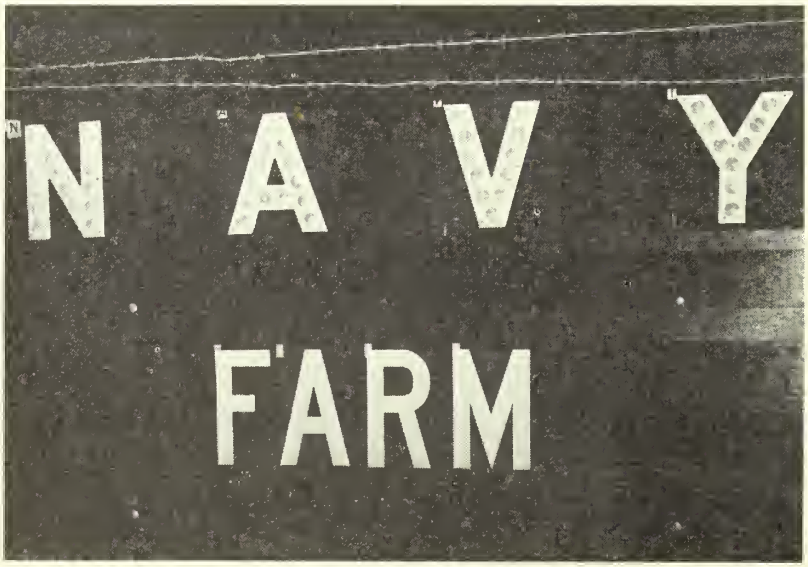

Figure 1.—Closeup view of the sign panel.

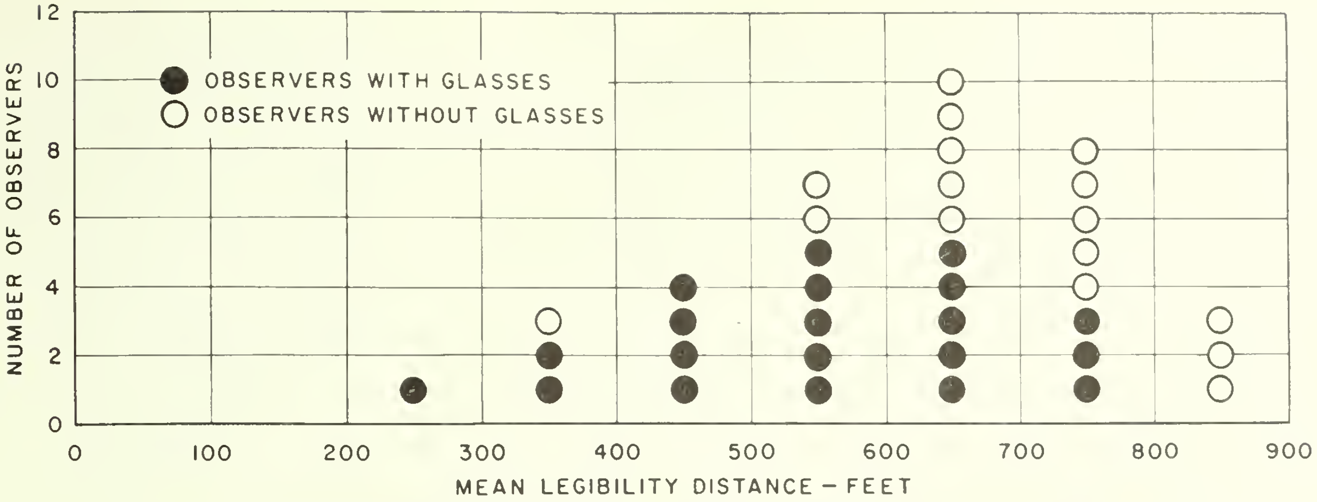

Figure 5.—Observations of 36 observers distributed according to the mean legibility distances.

to the Series E but of a different reflectorizing material, the mean legibility distance was 614 feet at normal spacing. Legibility increased to 657 feet at word lengths 40 percent greater than normal, but the 60-percent increase in word length resulted in a 13-foot decrease in the mean legibility distance.

As figure 6 clearly shows, Series E and Series ED alphabets should not be displayed at word lengths that are much more than 40 percent greater than normal, for at these wider spacings an actual decrease in legibility resulted. Little was gained by displaying the Series C alphabet at spacings beyond the 40-percent point.

The percentage increases in legibility with increased spacing were nearly identical for Series C and Series E alphabets, as shown by figure 7. An increase in word length of 40 percent over normal increased legibility 15 percent for Series C, and 16 percent for Series E, but only 7 percent for Series ED.

Effect of visual acuity

As noted earlier, highway signs should be designed for the driver with poor vision rather than for the average driver. The data were analyzed to determine whether drivers with poor, fair, and good eyesight showed similar and proportionate changes in their legibility patterns as letter spacings were increased.

Figure 8 shows mean legibility distances for each of the three visual acuity groups. In the case of the Series C alphabet, mean legibility distances for the poor visual acuity group were at least 250 feet below those of the good visual acuity group. For Series E and Series ED alphabets, the differences between the poor visual acuity group and the good group were over 300 feet. In general the patterns of legibility were similar for all groups.

These relations are shown on a percentage basis in figure 9. A 40-percent increase in word length resulted in a 12-percent increase in legibility of the Series C alphabet for the poor visual acuity group, 19 percent for the fair group, and 13 percent for the good group. For the Series E alphabet, percentage increases for the three groups were 17, 13, and 18 percent, respectively; while for Series ED the percentages were 6, 9, and 5 percent, respectively. The poor visual acuity group was within one percent of the average value for all three groups combined for Series E and ED alphabets, and within 3 percent for Series C. Because of this close agreement and the fact that relative values only are of principal importance in much of this study, the three groups were combined for most of the analysis.

After combining all visual acuity groups, the lower 15-percentile legibility distances at normal spacing were 350 feet for Series C, 400 feet for Series E, and 450 feet for Series ED. This compared closely with the mean legibility distances for the poor visual acuity group of 351, 414, and 449 feet, respectively.

One and two words per panel compared

When two words are placed on a single panel, each can have some effect on the legibility of the other. The 174 observations of words formed with the Series E alphabet and placed on the panel one at a time give a general indication of this effect, as shown by figure 10. The number of observations of "one word per panel" for a given visual acuity group and spacing was not the same. To correct for this, equal weight was given to each visual acuity group and spacing regardless of the number of observations within each category.

As the corrected curve in figure 10 shows, observations made with only one word per panel resulted in legibility distances ranging from 46 to 123 feet greater than for two words per panel. When only one word at a time was displayed, legibility showed a more rapid gain as spacing was increased, and continued to improve beyond the 40-percent increase in word length.

Legibility of six test words varies

Of the six words tested, STOP was consistently read at the greatest distance. Undoubtedly, this was due in part to the familiarity of this word to the average driver because of its use on the standard STOP sign. In addition, the letters are so arranged that a rather distinctive open shape results.

In the case of the Series C alphabet, BALK

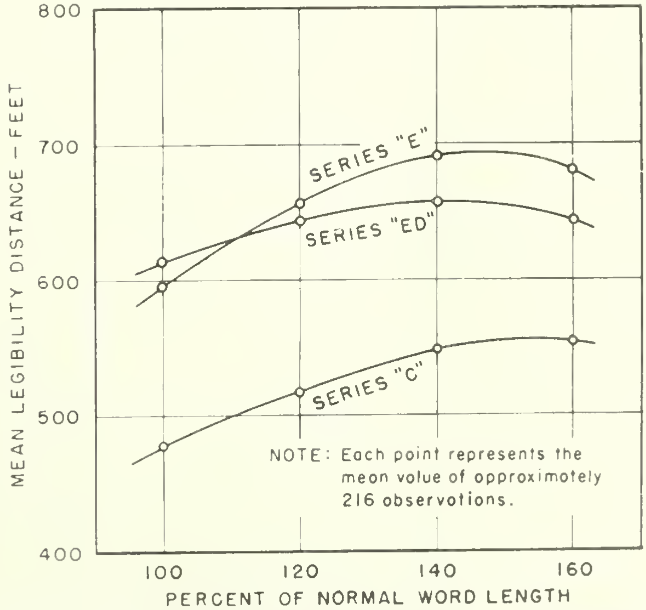

Figure 6.—Legibility distances for Series C, E, and ED alphabets as affected by the spacing between letters in test words.

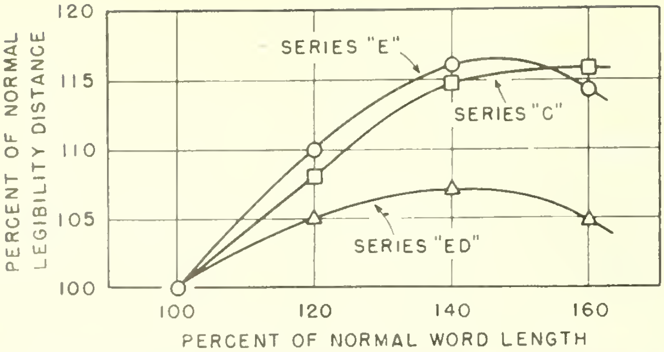

Figure 7.—Percentage change in legibility distances for Series C, E, and ED alphabets as affected by the spacing between letters in test words.

was the least legible at all spacings. The remaining words had about the same legibility, with DUCK slightly below the rest as shown in table 4. DUCK and BALK have somewhat similar shapes and the difficulty of distinguishing between them may have accounted for their diminished legibility. BALK was also near the bottom in legibility for both Series E and Series ED alphabets as shown in the table, while DUCK was slightly above average for Series E and slightly below average for Series ED.

Figure 11 shows the percentage increase in legibility of individual words with an increase in word length. The Series C alphabet had the most consistent pattern for the six words and Series ED, the most erratic pattern; the word FARM in letters of the Series ED alphabet actually showed a decrease in legibility as spacing increased.

Effect of panel position

The upper or lower position on the panel had little effect on legibility as shown by figure 12. Series C and E alphabets were both more legible when placed on the lower portion of the panel, approximately 5 feet above the roadway surface. The Series ED alphabet was more legible in the upper position, which was roughly 7 feet above the roadway surface. The average difference in legibility was generally less than 20 feet for any one of the three alphabet series, and has no significant implications for sign designers.

A possible explanation for the difference in legibility of words in the two panel positions is that the Series ED alphabet is made from a material with greater apparent reflectance than optimum for the low-beam illumination and other conditions of the test, while Series C and Series E alphabets are made from materials with less apparent reflectance than the optimum. Thus the upper panel position for the Series ED alphabet may in this case be better than the lower one because it receives less incident light from the headlamps and the apparent reflectance is less. With Series C and Series E alphabets the converse applies.

Comparison of Series E and ED Alphabets

One of the more interesting comparisons made involves the relative legibility values obtained with the Series E and ED alphabets. These were of similar design, except that the Series E alphabet was cut from reflective sheeting, and Series ED was constructed of plastic reflector units. As figure 6 and table 4 show, at normal spacing the Series ED alphabet was legible 18 feet farther than the Series E, while at word lengths 20 percent greater than normal, the legibility distance for Series ED was 10 feet less than Series E, and at word lengths 40 and 60 percent above normal, the legibility distances for Series ED were 34 and 37 feet less, respectively.

A characteristic of the letter design in the Series ED alphabet is that it results in slightly

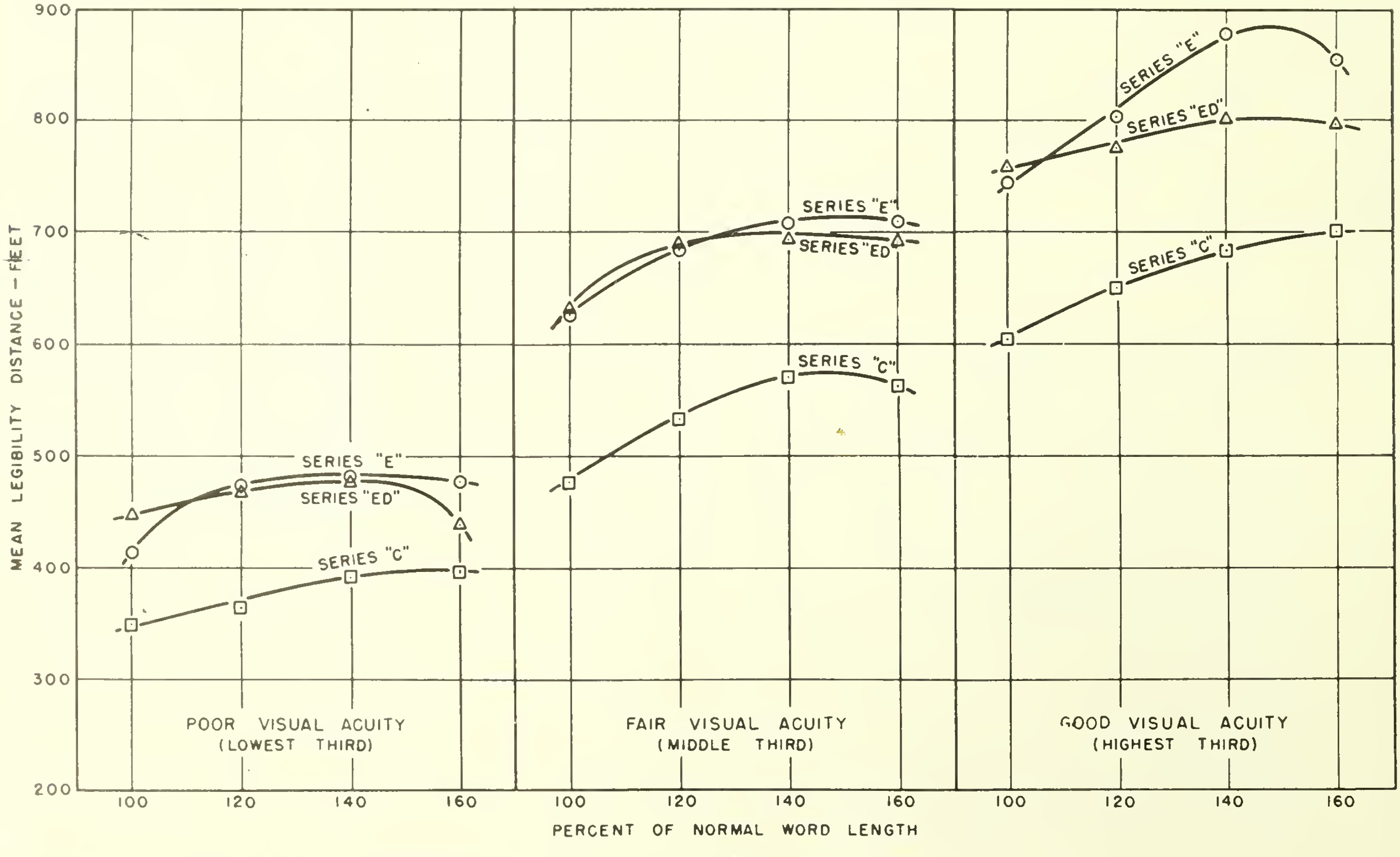

Figure 8.—Legibility distances, according to visual acuity groups, for Series C, E, and ED alphabets as affected by the spacing between letters in test words.

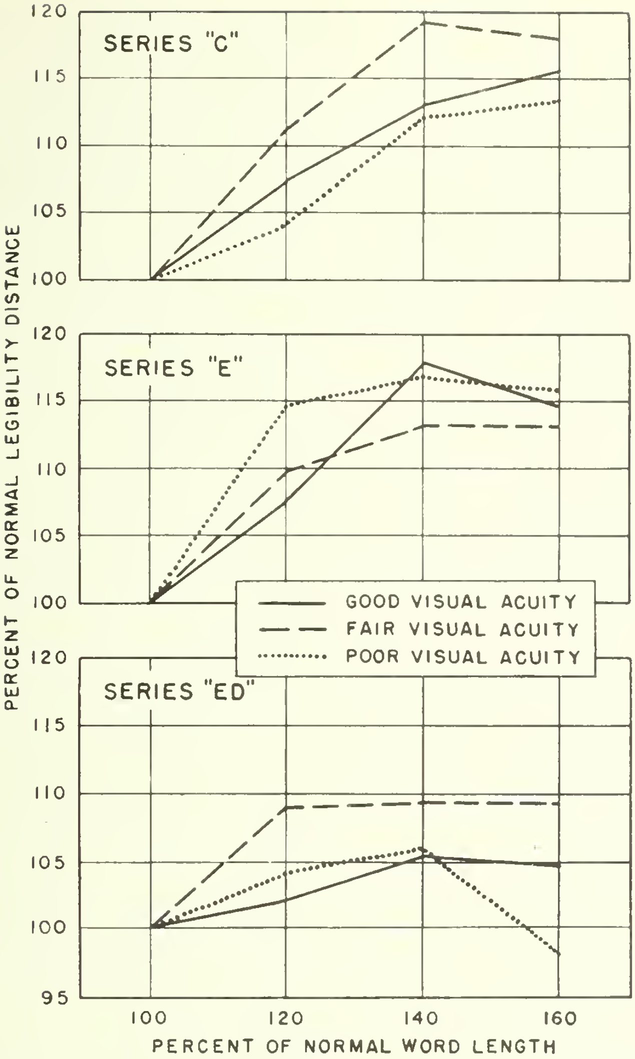

Figure 9 (Left).—Percentage change in legibility distances, according to visual acuity groups, for Series C, E, and ED alphabets as affected by the spacing between letters in test words.

greater word lengths than those of standard Series E. Figure 13 shows the mean legibility distance expressed in terms of feet of legibility distance per inch of word length. This tends to equalize the minor differences in word length between the two alphabets. Again the Series ED alphabet is superior at normal spacing and Series E at the wider spacings. The highest values, obtained with the Series C alphabet, are discussed later.

In the final analysis, the most useful value for comparative purposes is the height of letters of the Series E alphabet that results in a legend area per letter equal to Series ED at each of the four spacings. In order to obtain this, it was first necessary to know how the legibility of Series E changed with letter height.

A study reported by Forbes et al.[3] contains data that make possible this determination. That study used a white Series E alphabet on a black background. Night observations were made by individuals on foot and the signs were artificially illuminated. The spacing between letters was greater than the normal spacing of the present study and, in addition, the stroke width was about 16 percent greater than the standard. If the six words used in the present study had been similarly composed, the word lengths would have been 11 percent greater than normal.

The median legibility distances for 8- and 12-inch capital letters forming "place names with knowledge" were 690 feet and 1,060 feet, respectively. Thus, in this range of letter height, each inch increased the legibility distance an average of 92½ feet, and the calculated legibility of a 10-inch letter is 875 feet. In the present study, the corresponding median value was 640 feet, and the mean legibility distance was 630 feet.

The differences in results arise from several factors, principal among which are the driving observers, the display of two words per panel, and perception-reaction requirements. These factors are additive in their effect and help to account for the differences in legibility distances recorded. Because the results of these parallel studies are reasonably comparable, it is feasible to demonstrate the relative effects that letter height and spacing have on sign design.

The incremental value of 92½ feet per inch of letter height was derived from a word length 11 percent greater than normal in the present study. Proportionate ratios were computed for other word lengths, and these ratios were used to determine mean legibility distances of words formed with the Series E alphabet and measuring above and below 10 inches in height.

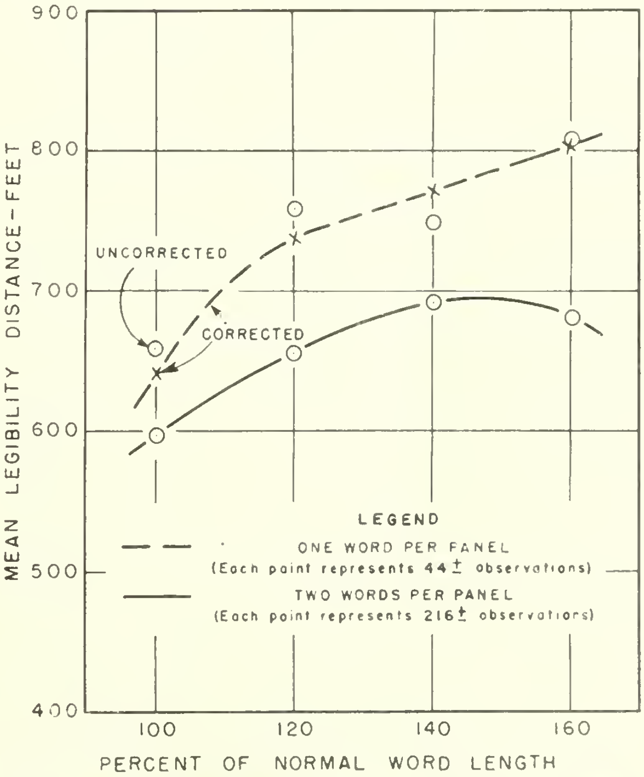

Figure 10.—Legibility distances, according to the number of words displayed on panel, for Series E alphabet as affected by the spacing between letters in test words.

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[3] *A comparison of lower case and capital letters for highway signs* by T. W. Forbes, Karl Moskowitz, and Glen Morgan. Proceedings of the Highway Research Board, vol. 30, 1950, pp. 355–373.

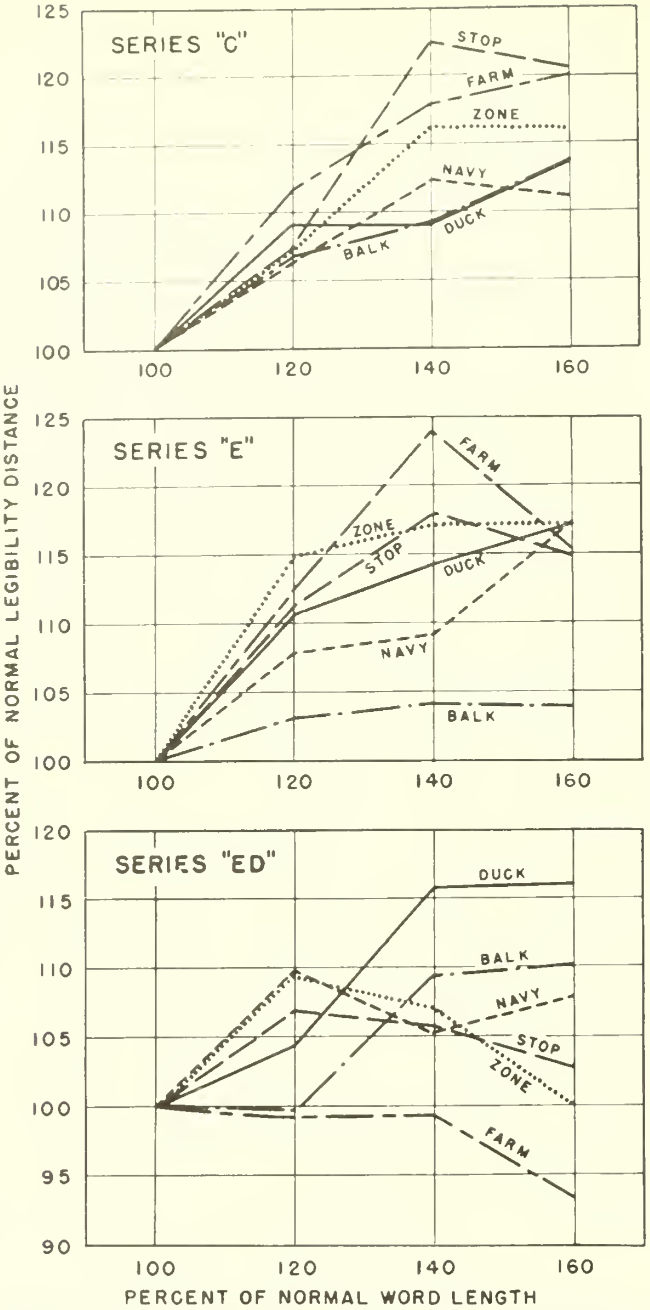

Figure 11.—Percentage change in legibility distances for each test word of Series C, E, and ED alphabets, as affected by the spacing between letters in test words.

For equal legend area per letter, a 10.05-inch letter of the Series E alphabet is equivalent to a 10-inch letter of the Series ED alphabet at corresponding spacings. At normal spacing the legibility of the 10-inch Series ED alphabet is 14 feet greater than the 10.05-inch Series E, while at word lengths 20, 40, and 60 percent greater than normal, the Series ED alphabet has 15, 39, and 42 feet less legibility.

Comparison of Series C and E Alphabets

As figure 6 shows, the Series E alphabet has an advantage over Series C of between 118 and 142 feet in legibility distance at the various spacings. The Series E alphabet occupies considerably more sign area, however, and if mean legibility distance per inch of word length is considered, Series C has the advantage as shown by figure 13.

Of greater interest, perhaps, is the resulting legibility when the two alphabets occupy equal legend area per letter and are displayed at the same relative spacing. A letter of the Series E alphabet, 8.34 inches high, occupies the same legend area as a 10-inch letter of the Series C alphabet at corresponding spacings. The mean legibility distances for the 8.34-inch letter height were calculated using the procedure outlined earlier.

The results of this calculation are shown in figure 14. It is seen that the Series C alphabet has an advantage over the Series E when legend area is taken into account. The differences are small, however, and since the curves were derived from two separate studies, a fairer statement would be that Series C and Series E alphabets are equally efficient users of sign space.

In a recent revision of the Manual on Uniform Traffic Control Devices,[4] it was recommended that the letters of the narrower Series A and B alphabets not be used for reflectorized signs. The present study, however, indicates that Series C, next in width, has no disadvantage when compared with the wider Series E alphabet. It is conceivable, therefore, that at least for night legibility of white reflectorized letters on a dark background, letters even narrower than Series C

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[4] *Revisions to the manual on uniform traffic control devices for streets and highways*, Bureau of Public Roads, 1954, p. 1.

Table 4.—Legibility distances for Series C, E, and ED alphabets as affected by the spacing between letters in test words

Test word	Mean legibility distance—feet														
	Series C alphabet					Series E alphabet					Series ED alphabet				
	Normal word length	Word length increased by— 20 percent	40 percent	60 percent	Average, all word lengths	Normal word length	Word length increased by— 20 percent	40 percent	60 percent	Average, all word lengths	Normal word length	Word length increased by— 20 percent	40 percent	60 percent	Average, all word lengths
STOP	526	564	644	634	592	638	710	753	734	709	661	706	699	680	686
NAVY	487	519	547	541	523	602	649	716	706	668	622	682	655	670	657
ZONE	480	515	559	558	528	568	651	666	666	638	629	689	674	629	655
DUCK	470	512	512	534	507	596	659	682	698	659	576	601	666	669	628
FARM	465	519	549	557	522	576	648	712	665	650	616	611	612	575	603
BALK	442	471	484	502	475	595	614	620	619	612	579	578	634	639	607
Average	478	517	549	554	---	596	655	691	681	---	614	645	657	644	---

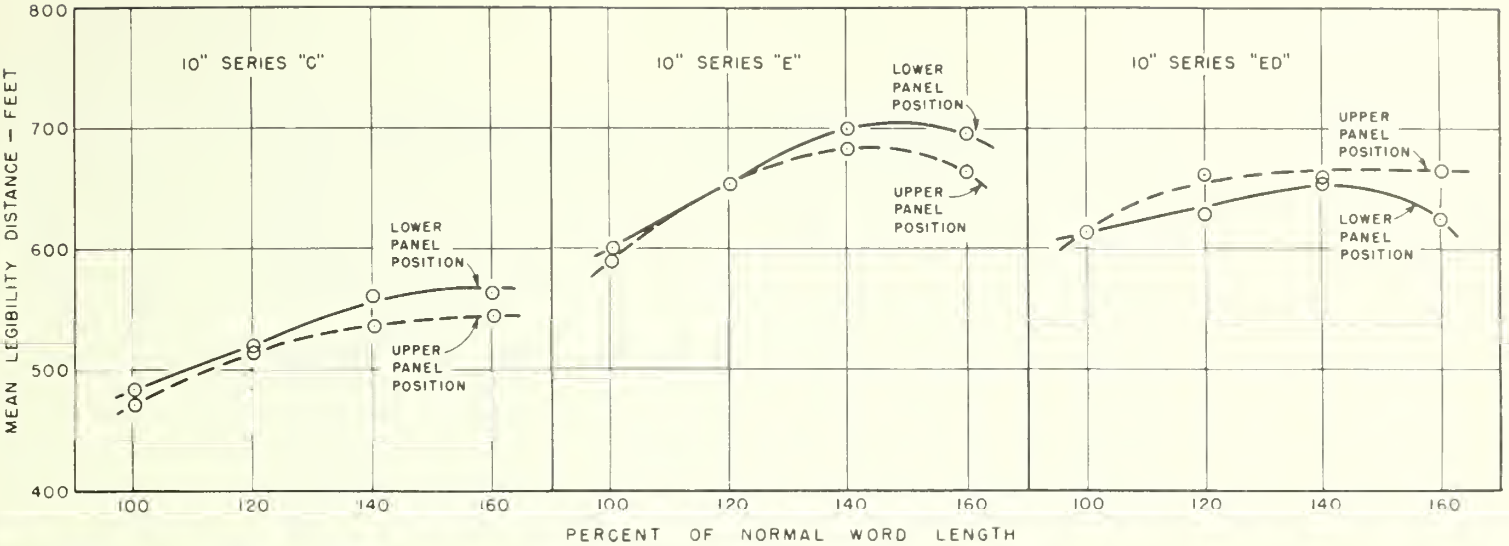

Figure 12.—Legibility distances, according to panel position, for Series C, E, and ED alphabets as affected by the spacing between letters in test words.

may also be as efficient as Series E. This finding may or may not hold for daytime viewing.

The effect of irradiation or spreading of the stroke width of white reflectorized letters may account for the close agreement in legibility of the 10-inch Series C and 8.34-inch Series E alphabets. A narrower stroke width for Series E might produce improved night legibility.

To obtain with letters of the Series C alphabet a legibility distance equivalent to that of Series E at comparable spacings, a greater letter height must be used. Words formed with the 10-inch Series C alphabet are only about as legible as those formed with the 8½-inch Series E alphabet. The legibility equivalent of a 10-inch Series E alphabet would be a Series C alphabet about 12 inches high. An obvious feature of this relation is the saving in the vertical dimension of a sign where short, wide letters are used in place of tall, narrow letters.

Alternative Methods of Improving Legibility

It has been shown that for a given alphabet, additional spacing between letters will increase legibility. The question still remains, however, as to whether legibility should be increased in this fashion or by increasing the letter height.

Again by use of the data contained in the Forbes study,[5] it is possible to compare the two alternatives, as in figure 15. Here it may be seen that a word, formed with 10-inch letters of the Series E alphabet with spacing 20 percent greater than normal, occupies about the same legend area per letter as a word with 11-inch letters at normal spacing, but gives slightly less legibility. An increase of 40 percent in word length increases the legibility distance somewhat less in proportion to that attainable by increasing the letter height. Beyond 40 percent, of course, legibility declines rapidly; thus the use of these wider letter spacings is undesirable.

In general, it is desirable that the initial sign layout be made at normal spacing, using the width of alphabet and letter height required for the legibility desired. In the development of a final design, the opportunity to introduce additional interletter spacing can often be used to advantage. Where two or more lines of sign copy are to be used, one line seldom fills the entire sign width. Increased spacing may be used for this line, thus increasing legibility with no increase in the size of sign panel required. For signs on overhead structures or for overhead signs at other locations, the vertical dimension of the sign may be limited and a comparatively small letter height may be required. In such instances, the common practice has been to shrink the total dimensions. Actually, the widest practicable alphabet (Series E or even the widest standard alphabet, Series F) and up to 40 percent more word length can be utilized to good advantage. Similarly, where the horizontal sign dimensions are restricted, letters of the narrower Series C alphabet can be employed at normal spacing, and legibility increased by using a greater letter height.

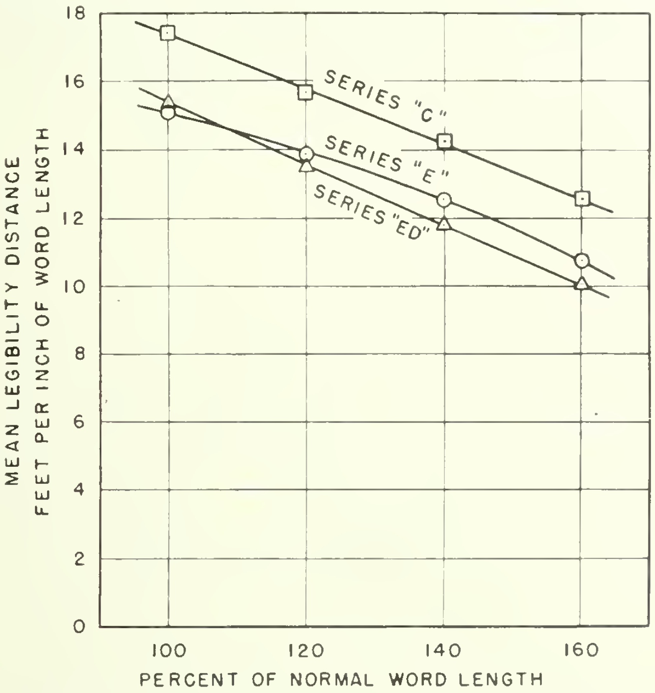

Figure 13.—Legibility distances (in feet per inch of word length) for Series C, E, and ED alphabets as affected by the spacing between letters in test words.

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[5] See footnote 3, p. 7.

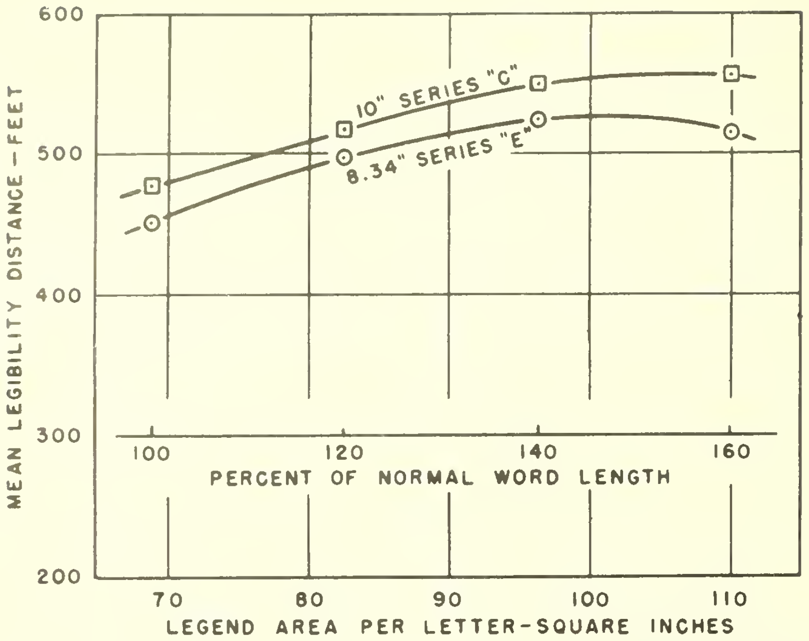


Figure 14.—Legibility distances for Series C and E alphabets (letters adjusted in height to produce equal legend areas) as affected by the spacing between letters in test words.

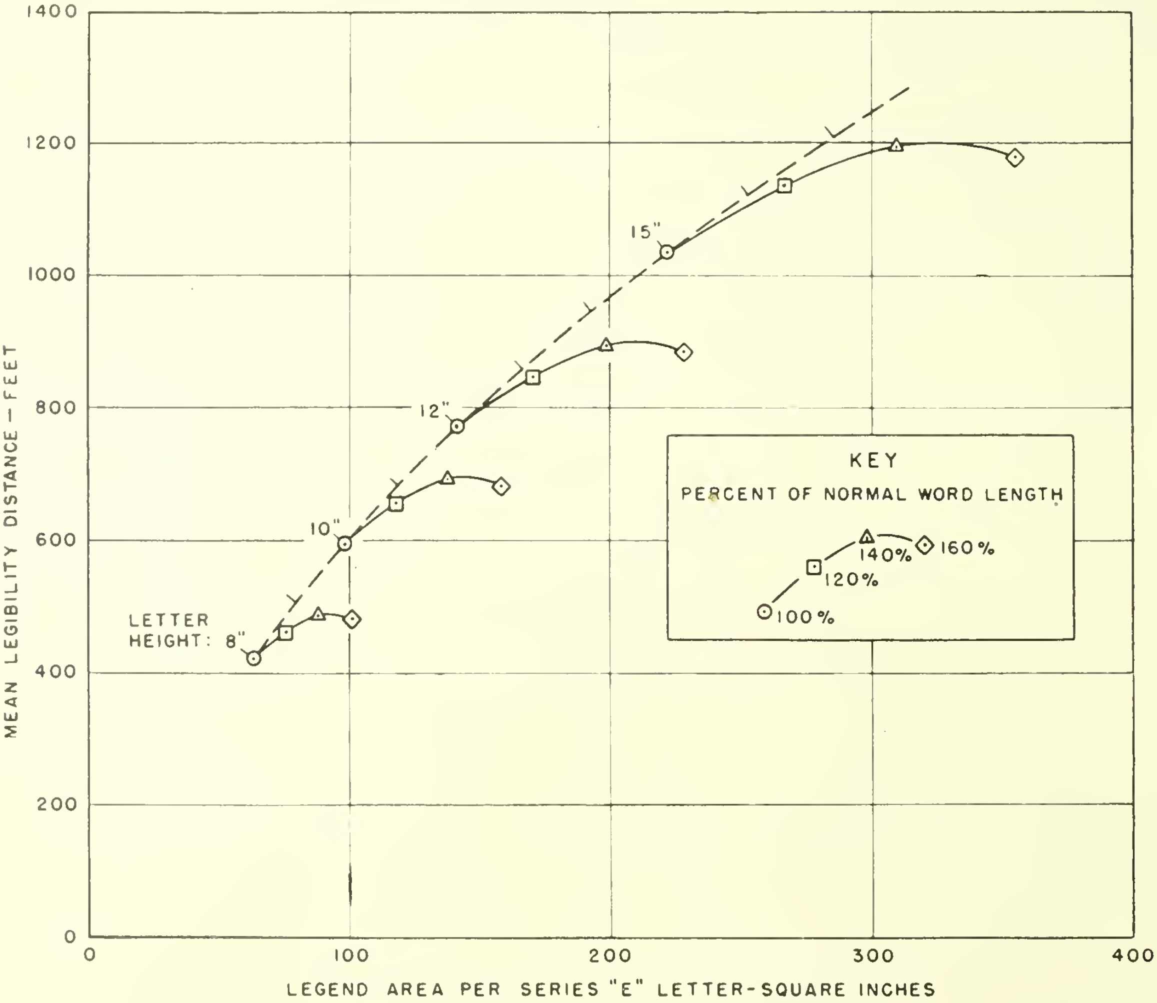
Application of Test Results

A sign found near many cities having large airports is AIRPORT, NEXT RIGHT. Figure 16 shows a possible design for such a sign. Series C and E alphabets are used with letters 12 inches high and normally spaced. These would probably be the desired proportions under normal conditions for a ground sign where adequate distance is available for advance warning and the average running speed is about 50 miles per hour.

With expressways handling heavy volumes of traffic, overhead signs are often necessary. Vertical clearances might be restricted if the sign were placed on an overhead structure. In that event, the sign could be stretched out as shown in figure 17. The vertical dimension then is reduced from 4 to 3 feet, and the horizontal dimension increased from 8 to 10 feet.

AIRPORT is comprised of letters of the Series F alphabet, 10 inches high, and the spacing is increased so that the word length is 40 percent greater than normal. This results in a word that has substantially the same legibility as the ground sign, which had 12-inch letters of the Series E alphabet and normal spacing. Similarly, NEXT RIGHT uses 10-inch letters of the Series E alphabet at a word length 20 percent greater than normal. The resulting legibility is better than could be obtained from the ground sign, which had 12-inch letters of the Series C alphabet and normal spacing.

Figure 15 (Below).—*Legibility distances for Series E alphabet as affected by the height of letters and the spacing between letters in test words.*



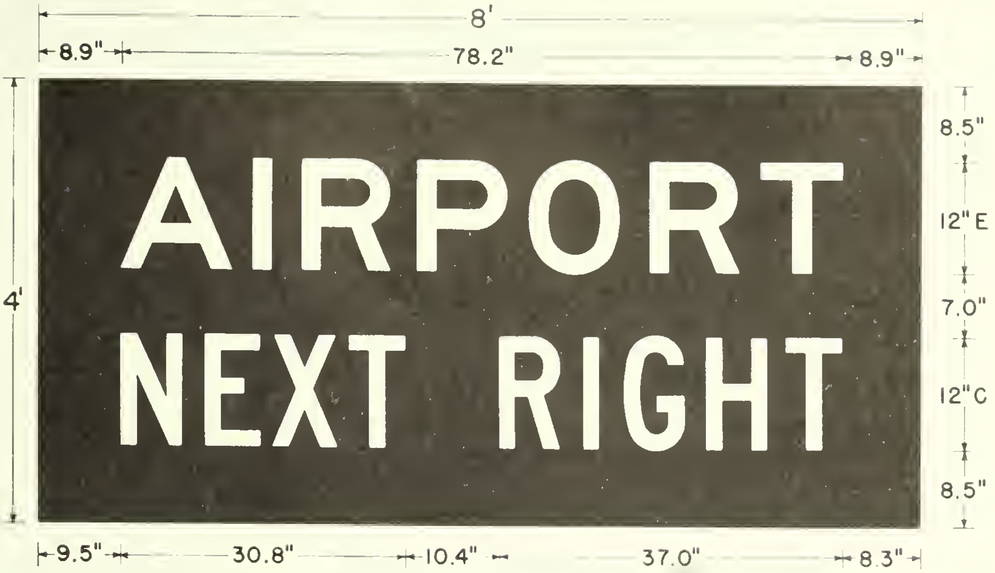


Figure 16.—Typical ground sign with 12-inch letters and normal spacing.

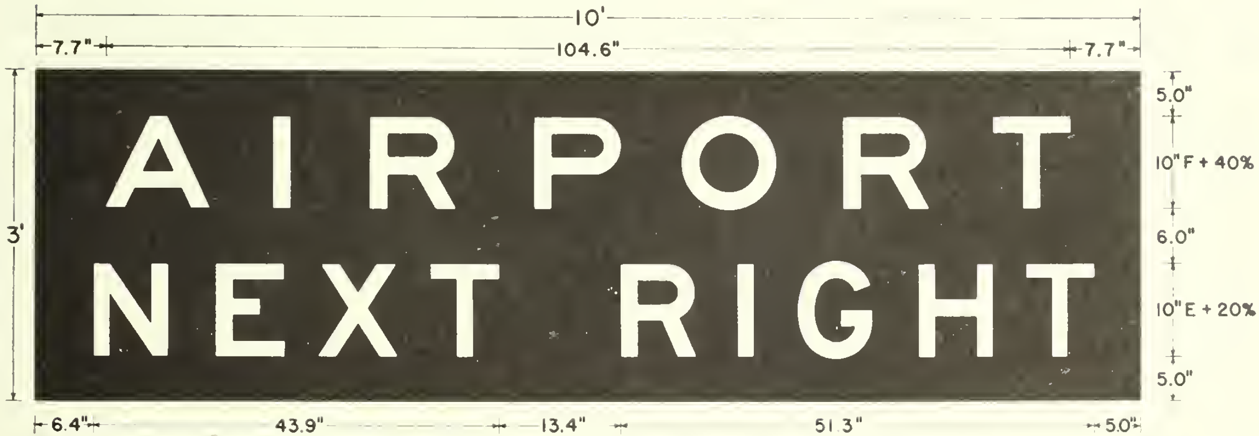


Figure 17.—Overhead sign with 10-inch letters and extended spacing.

New Publication

The Bureau's HIGHWAY STATISTICS, 1954, the tenth of the bulletin series presenting annual statistical and analytical tables of general interest on the subjects of motor fuel, motor vehicles, highway-user taxation, financing of highways, and highway mileage is now available.

The 135-page publication may be purchased from the Superintendent of Documents, Government Printing Office, Washington 25, D. C., at 75 cents a copy. The series of the annual bulletins that are available from the Superintendent of Documents are indicated on the inside back cover of PUBLIC ROADS.

Tomorrow's Highways Depend Upon Today's Law

By DAVID R. LEVIN [1]
Chief, Land Studies Section
Bureau of Public Roads

IT bids well for the Nation that women are becoming seriously interested in better and safer highways, and want to do something about it. A tenacity of purpose and a never-ending dedication to an ideal have always characterized such an interest. These very sessions on traffic safety provide the evidence.

A new era is emerging in highway history. It might be called the era of modernization. Unprecedented traffic volumes, increasing urbanization, and the rapidly mounting inadequacy of the highway plant compel consideration by all States and localities of the need for large-scale highway improvement.

Highway modernization is not accomplished by magic. Financial resources must be provided to do the job. Engineering and administrative talent must be enlisted to bring the latest technological advances to bear on the problem. Finally, we must provide the very best laws to support all of these efforts. In fact, unless we have the best legal tools we can forge, the public official will be powerless to improve the safety of our highways.

Many of the laws under which State and local highway officials now have to work are of the horse-and-buggy vintage. In the highway field, we are still trying to fly a jet plane with a "Model T" motor. It just can't be done without frightful consequences, many of which have already been enumerated for you by previous speakers.

Let me give you a few illustrations of what I'm talking about. While the most qualified engineering opinion points to the expressway (with control of access) as the best single answer to moving large volumes of vehicular traffic safely and efficiently, five States still have no specific statutory authorization for control of access. Many of the remaining States which do have laws on this subject have only inadequate authorizations.

Even isolated parcels of land needed for highway right-of-way purposes can effectively block construction progress. In such instances, it is in the public interest for public authority to have the right of immediate possession, with full protection of the property owner's rights to just compensation. Yet even today, almost one-half of the States are unable to obtain possession of highway rights-of-way when the need is greatest.

In some States, there remain in full force and effect on the statute books such prize specimens as the provision for a fine of $25 for the offense of leaving horses attached to a carriage carrying passengers for hire, and for not leaving the reins in the hands of some person to prevent the horses from running away.

The chief counsel of the Michigan State Highway Department and the assistant attorney general of that State recently reported that, of some 1,000 provisions of law relating to highways (a number of which date back to 1883), roughly one-third of them are obsolete, outmoded, in conflict with subsequent legislation, and are otherwise meaningless. Under such circumstances even the most able lawyers disagree as to what is the law. Little wonder that the highway engineer becomes concerned when he is sincerely trying to build a highway system.

Extensive modernization of the highway plant may mean that houses and other structures which are found within the proposed right-of-way limits must be moved or demolished, and that appropriate accommodations must be found for the occupants. This is particularly true in the urbanized areas, where modernization of highways is most critically needed. Yet the State laws designating procedures for rehousing and tenant relocation are found only in a few States.

Because of the rapid increase in the number and use of motor vehicles in the United States, right-of-way widths which were adequate yesterday are obsolete today. Yet a number of State laws—some of them in the Eastern States—specify that highway right-of-way width cannot exceed a designated amount; a width that was adequate for a transportation era that has long passed.

A number of States have evolved long-range highway programs; some of which stretch over a period of 10, 12, 15 years, and longer. The Congress of the United States, even now, has before it some important proposals for an accelerated highway program to take place over the next 10, 12, or 15 years. In all of these long-range programs, it is essential that the lands and property, which will be needed for the entire system, be acquired considerably in advance of need. A review of the highway laws in force today reveals that only 14 States can acquire lands for future use, and many of these laws are not entirely adequate. Incidentally, enormous savings are possible in programs of advance land acquisition. The California Division of Highways has estimated that it can save from $7 to $30 in the next 12 years for every dollar it invests today in highway rights-of-way needed for the future.

I could go on and on painting a darker picture with each additional illustration. But lest we become too discouraged, let me hasten to add that there is now sufficient reason to believe that the highway laws situation is going to improve, and rapidly.

In order to cope with this situation, the American Association of State Highway Officials has asked the Highway Research Board to set up a highway laws research project. A staff of five attorneys and three stenographers are engaged on this project. Incidentally, I am very happy to tell you that one of the five lawyers is a very brilliant woman.

The project is to take approximately 3 years to complete. The objectives of the study are twofold: (1) To ascertain, assemble, and analyze all existing State highway law in terms of logical, functional characteristics; and (2) with this background of fact, to isolate and determine what the important elements are of the laws in each functional category.

When this project is completed, we will know, with respect to traffic engineering or highway system classification, for example, what the laws of the 48 States now contain, and what the substantive elements are that should characterize any adequate statutory authorization in these fields. The same will be done for highway construction, land acquisition, maintenance, financing, drainage, and a host of other matters directly relating to highway development.

From this project and subsequent discussions there should finally emerge a cross section of the best thinking and experience as to what constitutes the basic elements in every functional phase of highway law. For the first time, there will have been developed a set of basic principles, which, by common consent, are deemed essential for adequate highway laws. With such practical yardsticks, every State will readily be able to evaluate its own body of laws in the light of present and future needs.

Now the all important question! What can you do to help? You can help a great deal. You constitute an important and articulate segment of the American public. As such, you can help tremendously by seeing to it that the highways in your community, county, and State are improved consistent with a sound, long-range plan. The program should be financed by a well-conceived fiscal policy and executed through highway laws that will help—not hinder—your highway officials who are trying desperately to do a job in the public interest.

Such an approach involves comprehensive factfinding to determine physical needs, fiscal requirements, and weaknesses in the highway law. You can help to generate interest in such investigations where needed. After they have been made, you can support the recommended findings with the same enthusiasm and vigor that has characterized your past program. The Committee on Highway Laws of the Highway Research Board will be pleased to make available to you the results of all of its research studies as they emerge.

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[1] This address was presented at the January 23, 1956, meeting of The General Federation of Women's Clubs, Eastern Region, Traffic Safety Forum, New York City.

Sawed Joints in Portland Cement Concrete Pavements, Progress and Problems

BY THE CONSTRUCTION BRANCH
BUREAU OF PUBLIC ROADS

Reported [1] by EDWIN J. COPPAGE, JR., Chief, Construction Management Section

SINCE the earliest days of concrete paving, joints have been formed in the plastic concrete by various means. Recently there has developed a new practice of cutting joints in the hardened concrete by means of special sawing equipment of various makes utilizing diamond or silicon carbide blades or discs. The cut is made only part way through the slab, creating a weakened plane which subsequently cracks through the full depth of the slab.

Purpose of Sawing Joints

The purpose of experimenting with sawed joints was to find a type of joint that would be smoother riding and less subject to spalling than the customary formed joints. Considerable difficulty had been experienced in many States in obtaining consistently good results with formed joints. The various processes employed to create a weakened plane, such as inserting and removing a steel bar and the hand tooling of the joints, apparently disturbed the plastic concrete in a way that weakened it and caused subsequent spalling. Also the finishing process often resulted in elevating the concrete slightly, creating bumps at the joints. The quality of a formed joint depends largely upon the skill of the workman making the joint. In contrast the sawed joint appears to have overcome these shortcomings even when the work is done by novices.

Spread of Experimentation

In the past 5 years the practice of sawing joints has spread rapidly over the country until at least 28 States have tried it and with minor exceptions have found the new practice to their liking. Sawed joints have been tried with apparent success upon a variety of pavement designs both reinforced and plain with slab lengths from 15 to 100 feet, both with and without load transfer devices of various designs. Kansas was the first State to make widespread use of sawed joints and was the first State to develop standard specifications. Relatively few other States have had sufficiently varied sawing experience to warrant the drafting of standard specifications.

Construction Problems

While sawed joints appeared to eliminate the spalling and roughness often associated with formed joints, they created new construction problems. It became apparent in the early sawed-joint projects that random transverse cracks were likely to occur unless so-called "control" joints or relief joints were either formed at intervals of 60 to 100 feet or sawed at an early age. Kansas initially elected to use formed control joints at 80 to 100 feet, thereby eliminating the problem of early sawing. With wet-earth curing to reduce shrinkage and 20-foot reinforced slab design, Kansas experienced no cracking between the formed control joints up to 30 days. Other States sawed intermediate joints at various times and little or no uncontrolled cracking was encountered.

Sawing Control Joints

There was a tendency for other States to follow the lead of Kansas and to form control joints in the plastic concrete rather than saw them. However, some believed that if sawed joints were superior to formed joints, it would be beneficial to have all joints sawed provided the sawing of control joints was feasible. Minnesota's experience showed that it was feasible to saw control joints, and that the best procedure to assure an adequate margin of safety between the time of sawing and the time of cracking was to saw the concrete at an age early enough to produce a slight amount of spalling and water erosion of the joint edges. Under certain favorable conditions such as (1) uniform temperature, (2) aggregates having a low coefficient of thermal expansion, and (3) curing methods having high insulating properties, the sawing of control joints may be deferred until the concrete has hardened sufficiently to prevent spalling and water erosion.

Experience in Virginia, Minnesota, Wisconsin, California, and Colorado on projects utilizing sawed-control joints indicates that concrete placed in the first part of the day or up until 12 or 1 o'clock has a tendency to crack much sooner than that placed in the afternoon. The morning concrete usually cracks the first night after placing, while the afternoon concrete generally cracks the second night. Consequently, it appears advisable to saw the control joints for morning concrete the same afternoon or night unless weather conditions retard the setting of the concrete to such a degree as to make sawing unfavorable.

The time required for the concrete to become hard enough to saw without excessive tearing has varied from 4 hours to over 24 hours. The most common way of determining when the concrete has hardened sufficiently to saw without excessive tearing is to make a very short cut with the saw and observe the condition of the joint edges. Other means are being sought to determine the earliest time when the concrete is hard enough to saw. Experience has shown that it is impractical to define the time for sawing as a specific number of hours elapsed after placing the concrete, due primarily to the influence of weather. Hot dry weather accelerates and cool damp weather retards the hardening of the concrete.

Some uncontrolled cracking has been experienced in a number of States where control joints were sawed rather than formed, but in practically all cases when the cracks had a tendency to occur prior to sawing the control joints the trouble was corrected by sawing at an earlier age. There was one exception to this general observation: control joints were being sawed in the early morning within the 24-hour specification limit for concrete placed the previous day and the slab was cracking through while the saw was only part way across the pavement, producing a random crack from the blade to the opposite edge of the slab.

This trouble was remedied by delaying the sawing for several hours. During this time the sun raised the slab temperature and reduced the tensile stresses to the point where sawing could be accomplished without danger of premature rupture. It is probable that earlier sawing in this instance, before shrinkage and contraction stresses approached the rupture point, would also have solved the difficulty. In similar situations where cracking is occurring during the sawing operation and temperatures are not expected to rise, it might be expedient to reduce the depth of saw cut.

Regarding the preceding discussion of control joints, it should be pointed out that the sawing was done with diamond blades which wear out much faster in cutting the green concrete than when cutting the intermediate joints in the older concrete. Consequently this system of sawing control joints in the green concrete and sawing the intermediate joints in the older concrete was preferable to consecutive sawing of all the transverse joints in the green concrete, not only because of the better quality of the joint edges but also for economic reasons. It is reported that, due to the sand particles working loose from the green concrete, blade life in sawing control joints is reduced to approximately one-half that in sawing intermediate joints.

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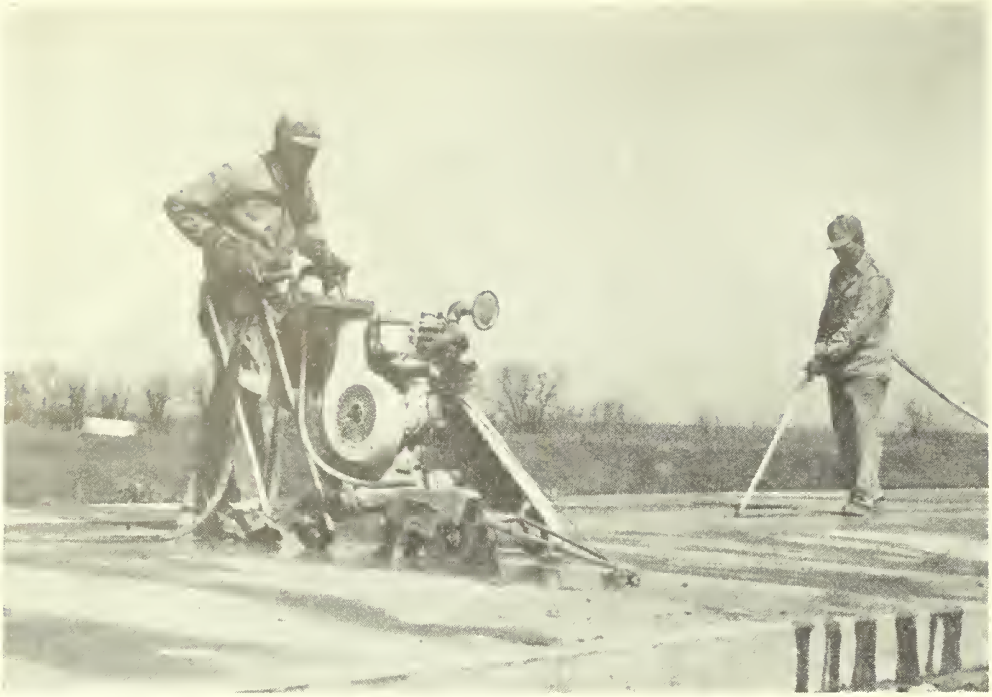
[1] This article was presented at the 35th Annual Meeting of the Highway Research Board, Wash., D. C., Jan. 1956.



Sawing operations of transverse joints and flushing of saw kerf from longitudinal joints.

Consecutive Joint Sawing

In the past 2 years increased use has been made of reinforced silicon carbide discs, in lieu of diamond blades, for sawing concrete made with the relatively softer aggregates such as limestone and slag. It is reported that these abrasive discs permit earlier sawing of the green concrete than diamond blades, and unlike diamond blades have a longer life in sawing green concrete than for sawing older pavement. Consequently the practice with silicon carbide discs has generally been to saw all joints early and consecutively.

Cost of Sawing

From an economical standpoint there appears to be a zone of aggregate hardness where the cost of sawing is approximately equal for diamond and silicon carbide discs. For relatively softer aggregates the silicon carbide is more economical and for harder aggregates the diamond is better. In some instances, silicon carbide discs were used for sawing the control joints and diamond blades were used to saw the intermediate joints.

The cost of sawing joints varies over a wide range depending upon a number of variables such as the type of aggregate, age or hardness of concrete mortar at the time of sawing, depth of cut, type of equipment, and skill of operators. Reported prices per linear foot range from a low of \$0.03 to a high of \$0.83, and average about \$0.35 per linear foot. The largest single item of expense connected with sawed joints is the blade cost, which for a 12- by ⅛-inch diamond blade averages about \$150. Reports on various experimental projects indicate that the blade life ranges from about 400 to 6,000 linear feet and averages approximately 1,500 linear feet. The cost of silicon carbide discs ranges from \$5 to \$18 per disc which is 10 percent or less of the cost of diamond blades.

Some attempt has been made to correlate blade life with aggregate hardness as measured by the percentage loss in the Los Angeles Rattler test, however the relation is inconsistent in some instances. Two States have reported that the presence of a small percentage of flint in the coarse aggregate caused silicon carbide discs to break. Apparently when a disc strikes a piece of the hard flint rock, it is deflected from a straight cut and an excessive bending movement causes the disc to shatter.

A comparison between the cost of forming and the cost of sawing joints is difficult due to the wide range in sawing costs. On several projects where aggregates were relatively hard, the cost of sawing was reported as approximately double that for forming joints. On numerous other projects, costs for sawing and forming were estimated to be equal. Certain benefits resulting from the sawing practice are difficult to evaluate, such as elimination of mixer shutdowns to allow the joint forming operations to catch up, and reduction of time between placing the concrete and applying the curing compound.

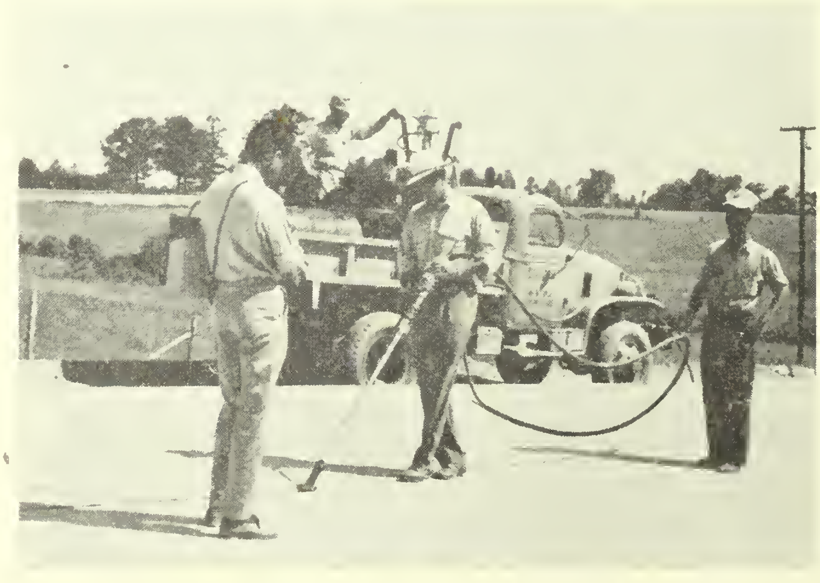
Sawing Longitudinal Joint

Some States have experimented with the sawing of the longitudinal joint as well as transverse joints. From their experience it has been found that the time interval before sawing is not critical, and the cost of sawing longitudinal joints is comparable with that for sawing intermediate transverse contraction joints.

Single-Lane Construction

Most of the pavements where sawed joints have been tried were placed full width or two lanes constructed at the same time, and the preceding observations apply to that type of construction only. Where a single lane is placed and the adjacent lane is constructed later, a serious construction problem is presented in placing and forming joints in the second lane. If the temperature drops after the second lane has been placed, the hardened concrete in the first lane contracts and joints in it that have cracked through open up. This movement is transferred to the green concrete slab by way of the tie bars and edge friction, thereby adding external stresses to the internal stresses caused by hardening, shrinkage, and temperature contraction.

California experimented with this type of construction and found that, at times, the initial lane transmitted enough stress to the second lane to result in an uncontrolled crack before the concrete had set hard enough in the latter to permit sawing. Subsequently other States had similar experiences, and some solved the problem by using formed joints in the second lane opposite every open joint in the first lane. Others, due to favorable local conditions such as relatively soft aggregates and moderate temperature fluctuations, have successfully eliminated uncontrolled cracking in



Sealing operation with cold-poured joint material.

this type of construction by very early sawing. The use of insulation applied to the surface of both lanes has not yet been reported by any State, but from theoretical considerations it would seem to be worth trying.

Design Problems

Joint depth

The sawed-joint technique has not necessarily created a new problem concerning the depth of cut, since the depth of a formed joint adequate to control the location of cracking will also be adequate depth for a sawed joint. There is one important difference. The cost of forming joints does not vary appreciably with the depth. Consequently, it has been customary to allow a generous safety factor in specifications for depth of formed joints. On the other hand, the cost of sawing varies pronouncedly with the depth. It is reported that a 2-inch cut costs over twice as much as a 1-inch cut.

Several States have varied the depth of cut from 1 to 2 inches on 8- and 9-inch slabs. Due to varying local conditions the 1-inch cut was found to be adequate in some States and inadequate in others. In all instances it was reported that 1½-inch depth of cut was satisfactory for both transverse and longitudinal joints.

Colorado reports that with 1-inch cuts in 8-inch slabs all the joints cracked through, but cracks of various lengths developed about 1 to 2 inches alongside of and roughly parallel to the saw cut. These cracks were from a few inches to several feet in length and would start and end at the saw cut. It is their belief that these cracks formed around the larger aggregate particles lying close to the surface. The maximum size of aggregate was 2 inches in this case. It is believed that aggregate size is a factor in determining the minimum depth of cut.

Joint spacing

The question as to the optimum spacing of joints has been present since the earliest days of concrete paving, and the advent of sawed joints has little effect upon the problem except as related to joint width and sealing.

Width of cut and sealing

While sawed joints may be cut the same width as formed joints, they generally are narrower—being about ⅛ inch when diamond blades are used and ¼ inch with silicon carbide discs. Wider joints may be obtained by making a second cut parallel to the first and usually shallower. The intervening concrete may be broken out with hand tools. Obviously this would materially increase the cost of jointing.

Until special equipment was made available, it was difficult to introduce seal material into the narrow ⅛-inch diamond blade cuts. After elimination of this difficulty, there still remained the question as to whether or not a ⅛-inch cut provided a reservoir for sufficient seal material to withstand the stretch to which it is subjected when the joints open in cold weather. On one project having 50-foot slabs, joints which were originally cut ⅛ inch measured ⅜ inch at near freezing temperature. This indicates a stretch of 200 percent for the seal material which is considerably above the 50 percent stretch commonly specified for seals.

Opinions vary regarding the importance of obtaining a perfect seal against moisture and foreign matter, possibly due to varying local conditions. Several States are experimenting with unsealed sawed joints; however available data are insufficient to warrant the drawing of conclusions. The gathering of data regarding joint openings has been greatly simplified with the advent of sawed joints. The smooth vertical faces of the saw cut make it relatively easy to measure joint widths by means of calipers or thickness gages, thus eliminating the necessity for casting reference plugs in the concrete as was formerly required.

Curing of Concrete

The curing method used is probably of greater importance for sawed-joint construction than for formed-joint construction. The effectiveness of the curing method with regard to water retention and insulating value affects the shrinkage stresses in the pavement, and is related to the time of sawing and joint spacing. Various types of cures have been used in conjunction with sawed joints, and apparently successful application of sawed-joint construction is not dependent upon the use of any one type of cure as was formerly believed by some highway engineers.

Sawing Equipment

A variety of sawing equipment is available from a number of manufacturers. There are manually pushed saws with single blades and self-propelled saws having one or several blades in tandem. The relative merits of the different types have not yet been evaluated. A separate water tank is usually required to provide ample water for cooling the blade and washing the saw kerf from the joint.

The WASHO Road Test—A Motion Picture

The Bureau of Public Roads has produced and released a motion picture, *The WASHO Road Test,* depicting the operation and major findings of the large-scale road test undertaken cooperatively by the Western Association of State Highway Officials, the Bureau of Public Roads, and the motor-vehicle and petroleum industries, and conducted under the direction of the Highway Research Board. The object of the test was to determine the effect of heavy traffic on bituminous pavements, specially built for the purpose near Malad, Idaho.

In essence, the test was planned to compare the performance of different designs of bituminous pavements, all laid on a uniform soil subgrade, under controlled truck traffic of varying axle arrangement and load. Traffic operations ran from November 1952 through May 1954. Analysis of the countless data obtained with a wide variety of tests and instruments, some newly created for the job, continued for more than a year thereafter. The design, construction, and testing procedures of the WASHO Road Test are fully detailed in HRB Special Report 18, and the test data, analyses, and findings in HRB Special Report 22, both published by the Highway Research Board, 2101 Constitution Ave., N. W., Washington 25, D. C.

The WASHO Road Test motion picture (16-millimeter, color and sound; running time 35 minutes) was produced by the Bureau of Public Roads as a visual summary of the published reports with the authorization and endorsement of the WASHO Advisory Committee of the Highway Research Board.

The WASHO Road Test film may be borrowed by any responsible organization upon application to the nearest office listed below. There is no charge for such loans except for the shipping costs. Loans can be made only for short periods of time. Several alternate dates should be proposed, and the request should be made well in advance of a planned showing.

WASHINGTON, D. C.: Research Reports Branch, Bureau of Public Roads, Washington 25, D. C.

SAN FRANCISCO, CALIF.: Division Engineer, Bureau of Public Roads, Room 102 Old Mint Bldg., 5th & Mission Sts., San Francisco 3, Calif.

PORTLAND, OREG.: Division Engineer, Bureau of Public Roads, 753 Morgan Bldg., 720 S. W. Washington St., Portland 8, Oreg.

Those who wish to purchase prints of the motion picture *The WASHO Road Test* should write to R. E. Royall, Chief, Research Reports Branch, Bureau of Public Roads, Washington 25, D. C., for authority to do so and for information on the purchase procedure. Prints will cost $141 each, but do not send your money to Public Roads. The request for authority to purchase prints must include the following statement:

"Assurance is hereby given that the composition of the motion picture will not be altered in any way, either by addition or deletion, and that it will be shown only in its entirety."

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Highway Capacity Manual (1950). $1.00.
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Highway Practice in the United States of America (1949). 75 cents.
Highway Statistics (annual):
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1948, 65 cents.
1949, 55 cents.
1950 (out of print).
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1952, 75 cents.
1953, $1.00.
1954, 75 cents.
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Highways in the United States, *nontechnical* (1954). 20 cents.
Highways of History (1939). 25 cents.
Identification of Rock Types (reprint from PUBLIC ROADS, June 1950). 15 cents.
Interregional Highways, House Document No. 379 (1944). 75 cents.
Legal Aspects of Controlling Highway Access (1945). 15 cents.
Local Rural Road Problem (1950). 20 cents.
Manual on Uniform Traffic Control Devices for Streets and Highways (1948) (*including 1954 revisions supplement*). $1.00.
Revisions to the Manual on Uniform Traffic Control Devices for Streets and Highways (1954). *Separate*, 15 cents.

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Mathematical Theory of Vibration in Suspension Bridges (1950). $1.25.
Model Traffic Ordinance (revised 1953). Out of print.
Needs of the Highway Systems, 1955–84, House Document No. 120 (1955). 15 cents.
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Public Utility Relocation Incident to Highway Improvement, House Document No. 127 (1955). 25 cents.
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United States System of Numbered Highways. 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

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DEPARTMENT OF COMMERCE - BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF FEBRUARY 29, 1956

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$7,949	$19,544	$10,688	230.9	$7,554	$4,155	42.9	$37,611	$19,830	599.6	$64,709	$34,673	873.4
Arizona	2,625	7,378	5,583	133.4	4,478	3,044	43.0	9,271	6,814	111.3	21,127	15,441	287.7
Arkansas	11,780	14,261	7,483	507.6	3,137	1,551	26.4	20,855	10,731	412.1	38,253	19,765	946.1
California	7,054	28,597	15,223	177.2	14,712	8,194	16.1	151,995	77,879	318.4	195,304	101,296	511.7
Colorado	16,452	6,750	3,737	98.4	3,938	2,310	43.2	14,772	7,787	158.4	25,460	13,834	300.0
Connecticut	22,472	1,440	825	5.5	702	347	1.1	11,637	5,791	14.9	13,779	6,963	21.5
Delaware	5,101	1,909	974	6.2	2,563	1,303	42.1	4,859	2,434	28.0	9,331	4,711	76.3
Florida	8,146	15,541	8,365	194.3	12,848	6,706	60.2	33,901	17,359	343.7	62,290	32,430	598.2
Georgia	21,633	26,351	13,598	418.2	5,241	2,481	43.2	43,284	20,366	713.0	74,876	36,445	1,174.4
Idaho	6,794	6,685	4,329	92.0	1,389	966	32.1	12,427	8,124	149.2	20,501	13,419	273.3
Illinois	15,963	36,460	19,742	522.8	33,140	18,033	99.6	81,182	43,922	375.0	150,782	81,697	997.4
Indiana	27,043	22,407	11,435	235.0	15,763	7,909	104.7	38,117	21,294	89.0	76,287	40,638	428.7
Iowa	8,785	22,839	13,152	783.6	13,366	6,815	198.3	17,668	9,547	713.7	53,873	29,514	1,695.6
Kansas	10,967	14,672	7,654	864.2	5,557	2,921	103.8	21,349	11,002	768.6	41,578	21,577	1,736.6
Kentucky	15,452	4,429	2,430	42.2	2,554	1,694	2.7	38,690	20,172	585.3	45,673	24,296	630.2
Louisiana	9,799	15,187	7,608	94.2	6,231	3,115	4.5	44,583	21,192	485.8	66,001	31,915	584.5
Maine	7,781	2,974	1,555	26.6	86	86		15,228	7,859	107.9	18,288	9,500	134.5
Maryland	5,757	25,777	13,232	73.2	11,314	5,493	14.4	16,451	8,706	61.2	53,542	27,431	148.8
Massachusetts	13,570	12,143	6,062	18.5	19,353	10,220	20.6	36,470	17,567	50.5	67,966	33,849	89.6
Michigan	12,299	40,258	20,340	653.4	21,692	12,198	102.6	53,198	27,001	369.0	115,148	59,539	1,125.0
Minnesota	11,026	17,328	8,735	929.6	10,504	6,022	197.5	27,075	14,452	507.8	54,907	29,209	1,634.9
Mississippi	12,260	11,845	6,001	446.4	3,770	1,983	142.6	25,974	13,334	620.8	41,589	21,318	1,209.8
Missouri	7,795	27,281	13,957	1,179.3	7,314	3,895	29.3	75,788	39,695	1,154.8	110,383	57,547	2,363.4
Montana	17,392	9,391	5,804	172.9	4,212	2,446	80.2	22,821	14,299	386.7	36,424	22,549	639.8
Nebraska	13,549	18,557	9,682	784.0	8,072	4,179	163.4	23,783	12,764	709.1	50,412	26,625	1,656.5
Nevada	11,439	2,330	1,992	22.5	2,349	1,940	38.7	6,578	5,567	146.8	11,257	9,499	208.0
New Hampshire	6,277	2,633	1,406	14.0	595	300	2.4	8,395	4,322	52.7	11,623	6,028	69.1
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New Mexico	7,190	4,120	2,596	43.2	3,995	2,601	75.3	10,935	7,222	178.2	19,050	12,419	296.7
New York	52,667	32,138	16,642	112.6	49,688	25,441	85.9	200,936	93,100	242.5	282,762	135,183	441.0
North Carolina	13,738	20,389	10,163	417.3	6,460	3,141	62.1	47,035	23,227	586.6	73,884	36,531	1,066.0
North Dakota	5,554	15,092	7,685	1,419.7	6,585	3,425	481.5	8,873	4,535	491.7	30,550	15,645	2,392.9
Ohio	25,770	46,890	24,092	175.6	17,491	9,256	29.8	67,273	32,545	102.6	131,654	65,893	308.0
Oklahoma	14,873	16,359	8,219	393.3	11,443	5,978	101.0	36,007	18,901	359.1	63,809	33,098	853.4
Oregon	4,597	4,231	2,533	60.8	3,298	2,017	32.5	23,822	14,695	221.6	31,351	19,245	314.9
Pennsylvania	30,229	21,650	11,178	73.3	59,505	29,179	64.5	84,345	42,746	311.3	165,500	83,103	449.1
Rhode Island	1,647	4,010	2,010	13.2	1,386	693	1.0	19,575	10,100	28.9	24,971	12,803	43.1
South Carolina	11,384	16,333	8,856	352.8	2,496	1,321	31.2	18,087	9,455	329.7	36,916	19,632	713.7
South Dakota	1,296	23,869	13,712	925.6	6,393	3,775	199.5	12,782	7,373	562.2	43,044	24,860	1,687.3
Tennessee	17,740	18,029	9,021	523.1	10,476	5,241	70.4	40,135	18,265	329.6	68,640	32,527	923.1
Texas	19,793	27,176	13,806	637.1	24,957	13,322	201.4	110,234	58,082	1,622.0	162,367	85,210	2,460.5
Utah	5,030	4,978	3,723	160.1	1,279	977	19.7	12,032	9,017	126.7	18,289	13,717	306.5
Vermont	4,503	2,927	1,464	14.4	914	444	2.4	8,244	4,314	87.0	12,085	6,222	103.8
Virginia	16,186	17,175	8,825	261.7	5,516	2,912	26.6	26,221	12,888	306.7	48,912	24,625	595.0
Washington	6,777	23,418	13,128	354.0	3,176	1,764	71.1	21,990	11,696	172.2	48,584	26,588	597.3
West Virginia	16,368	10,023	5,271	43.0	2,602	1,319	4.3	17,470	8,770	65.1	30,095	15,360	112.4
Wisconsin	13,725	17,689	8,905	251.7	8,720	4,744	80.6	44,435	22,004	415.8	70,844	35,653	748.1
Wyoming	337	10,602	6,984	213.9	1,960	1,294	48.6	13,568	8,896	232.2	26,130	17,174	494.7
Hawaii	4,517	2,799	1,390	8.0	3,177	1,571	2.9	5,575	2,547	12.8	11,551	5,508	23.7
District of Columbia	9,574	4,492	2,246	3.4	384	191	.5	9,728	4,392	3.3	14,604	6,829	7.2
Puerto Rico	11,637	4,672	2,277	24.8				15,789	7,213	53.1	20,461	9,490	77.9
TOTAL	633,273	780,193	414,216	15,282.9	459,711	243,256	3,357.1	1,779,544	916,059	16,910.5	3,019,448	1,573,531	35,550.5

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In this issue: A study of service lives of highway surfaces. How long will a surface remain in use before it is resurfaced, reconstructed, or otherwise replaced?

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C. M. Billingsley, *Editor*

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Lives of Highway Surfaces—Half Century Trends

BY THE FINANCIAL AND ADMINISTRATIVE RESEARCH BRANCH BUREAU OF PUBLIC ROADS

Reported [1] by GORDON D. GRONBERG, Head, Annual Cost Unit, and
NELLIE B. BLOSSER, Statistician

That highways do wear out is an accepted fact. Since World War II our construction program has been greatly expanded. What influence has this stepped-up program had on the service lives of our roads? Are the miles of roads actually retired greater or lesser than those developed in past estimates? This article attests to the validity of previous estimates and adds 7 years experience to the information presented in an earlier article.

Among the major objectives of the road-life study phase of the highway planning surveys is the development of factual data relating to (1) how long road surfaces remain in service before they are replaced, and (2) what is done to the road surfaces at the time of replacement. This is the third article presented on this subject in the past 16 years. The first article was published in 1941 and the second in 1949.

In the present article the life experience of road surfaces on primary rural highways covers the period from January 1, 1900, to January 1, 1953. The basic data were submitted by 25 States and Puerto Rico for rural State or Federal-aid primary systems and include 344,108 miles of construction and 192,741 miles of retirements. Results of the analysis show that the number of years a surface remains in service before it is resurfaced, reconstructed, or otherwise replaced ranges from 5.2 years for lower type surfaces to 25.5 years for the higher type surfaces.

Of the 183,976 miles analyzed by method of retirement through 1952, 57 percent were resurfaced, 31 percent reconstructed, 9 percent transferred to other public agencies, and 3 percent abandoned. Since the end of World War II the proportion resurfaced has decreased and the proportion reconstructed has increased.

Data for nine States are common to all three articles. For these nine States, an estimate was made of mileages that would still be in service in 1953 if road surfaces continued to be retired at the rates shown in the 1941 and 1949 studies. This estimate was then checked against the actual mileages remaining in 1953. There was close agreement between the actual and estimated miles remaining. The differences between the previous forecasts and the present study may be partly due to the lagging highway program during the war and postwar periods.

The service life data developed in this article were used in estimating the probable mileages remaining in service in future years. During the 10 years, January 1, 1953, to January 1, 1963, it is estimated that 96 percent of the low-type, 83 percent of the intermediate-type, and 61 percent of the high-type surfaces in service at the beginning of the period will be retired through resurfacing, reconstruction, abandonment, or transfer.

THE highway systems of the Nation are a vital segment of our national economy. The proper management of these systems and the protection of the investment they represent require detailed knowledge of their performance. Many facts are needed with respect to their construction, maintenance, operation, and administration. To supply these facts is one function of the highway planning surveys which were established in the middle 1930's by the State highway departments in cooperation with the Bureau of Public Roads.

Despite their extent and advanced stage of development most of our highways are inadequate for the demands of present day traffic. Much of the current inadequacy dates to World War II when highway construction was cut to a minimum as a result of the defense effort. Highways were kept in operating condition with a minimum of expenditure to conserve labor, equipment, and materials. At the end of World War II an out-dated highway system was called upon to handle heavier traffic demands than ever before. To solve the problem, many States undertook highway needs studies. Attention was directed to both long- and short-range planning.

Funds were limited and progress was slow. In the meantime, the needs continued to mount. There was need for current information relating to the performance of roads; that is, how fast they were wearing out and how long they would remain in service. The proper evaluation of these data is necessary to determine the dimensions of future needs. The data necessary for these evaluations are obtained from the road life studies of the highway planning surveys. Included are such items as rates of wearing out, construction cost, maintenance cost, extent of functional obsolescence and structural deterioration, and life of the investment. The findings must be appraised in the light of changing traffic volumes, heavier loads, and higher speeds.

One of the objectives of road life studies in the individual States is the development of an organized body of information concerning life characteristics of highway surfaces. The first comprehensive analysis of such data was published in 1941.[2] In that article (hereafter referred to as the 1941 article or study) were included the results of service-life analyses on 210,000 miles of construction up to January 1, 1937, for various surface types in 26 States.

There was little activity in the road life studies from 1940 through World War II. However, most States resumed operations once the war was over and a second article on service lives of roads in 16 States was published in 1949.[3] Included in the study (hereafter referred to as the 1949 article or study) were the service-life analyses on 248,783 miles of construction and 129,593 miles of retirements up to January 1, 1946.

In the present article, data for 25 States and Puerto Rico (see fig. 1) are included:

Arizona
California
Connecticut
Delaware
Florida
Georgia
Illinois
Indiana
Kansas
Minnesota
Mississippi
Missouri
Montana
Nevada
New Mexico
Oklahoma
Pennsylvania
Rhode Island
South Dakota
Tennessee
Texas
Washington
West Virginia
Wisconsin
Wyoming
Puerto Rico

Nine of these States were also included in the 1941 and the 1949 articles: Indiana, Kansas, Missouri, Montana, New Mexico, Oklahoma, Texas, West Virginia, and Wyoming. In addition to the nine States, the

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[1] This article was presented at the 35th Annual Meeting of the Highway Research Board, Wash., D. C., Jan. 1956.

[2] *Life characteristics of surfaces constructed on primary rural highways*, by Robley Winfrey and Fred B. Farrell. PUBLIC ROADS, vol. 22, No. 1, Mar. 1941; also Proceedings of the Highway Research Board, vol. 20, 1940, pp. 165–199.

[3] *Life characteristics of highway surfaces*, by Fred B. Farrell and Henry R. Paterick. PUBLIC ROADS, vol. 25, No. 9, Aug. 1949; also Proceedings of the Highway Research Board, vol. 28, 1948, pp. 40–52.

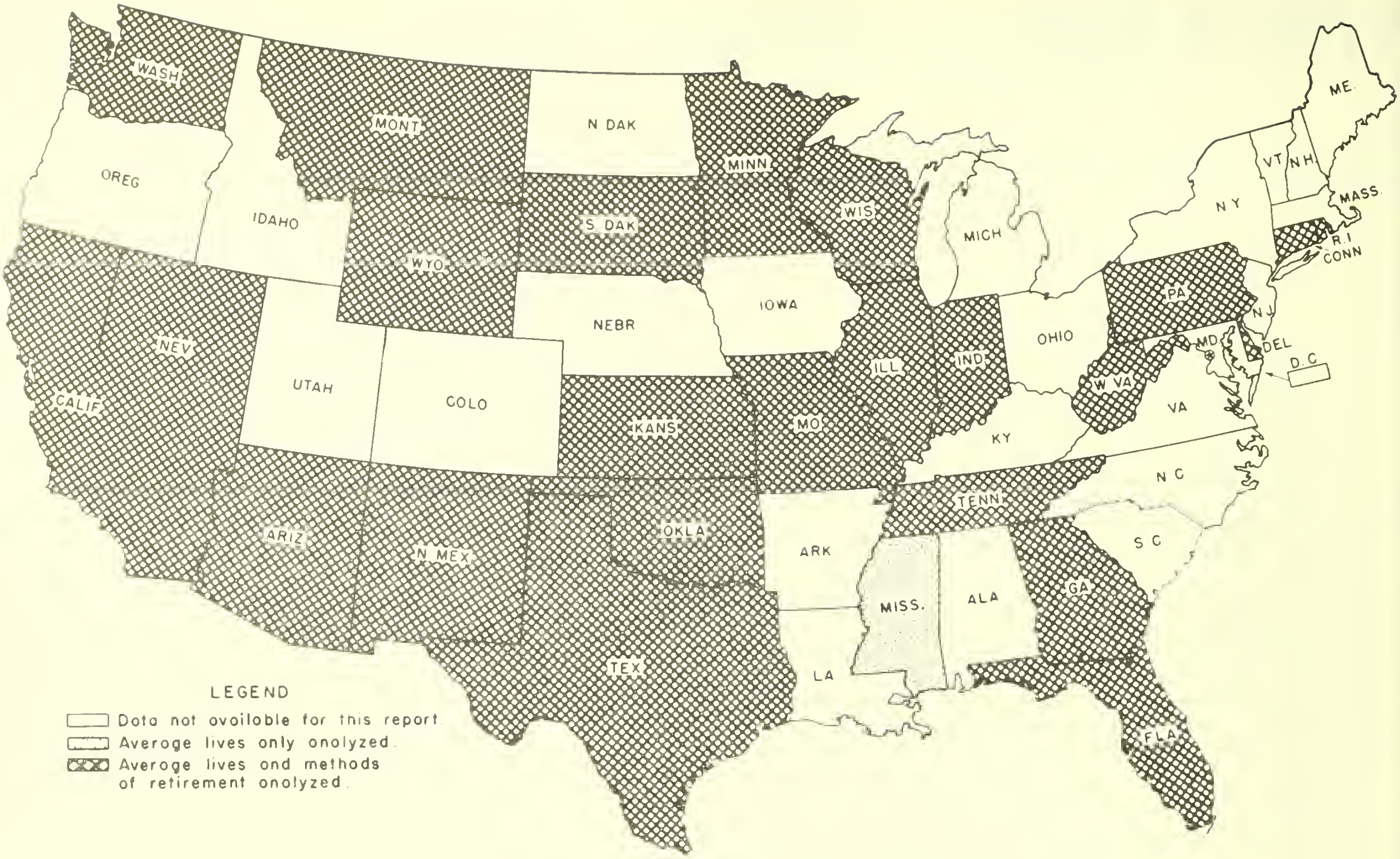

Figure 1.—States for which road-life mileage data are included.

present article includes four States listed in the 1949 study: Illinois, Minnesota, Nevada, and Wisconsin.

Basic Data Compiled

This article embraces 344,108 miles of construction of various surface types on the rural portions of the primary State or Federal-aid systems for 25 States and Puerto Rico. In general, all mileage in incorporated places of more than 1,000 persons has been excluded. Construction involving widening of roadways has also been omitted in those cases where the widening was done as a separate operation. Where the widening was done in conjunction with the resurfacing or reconstruction of the previous surface, the mileage of the new construction, which includes widening, is represented in the tabulations. The basic tables 1 and 4 cover the period from January 1, 1900, to January 1, 1953, and summarize the following information:

1. Mileage constructed each year for each surface type (for 25 States and Puerto Rico).
2. Mileage of each year's construction of each surface type remaining in service on January 1 of each year after construction (for 25 States and Puerto Rico).
3. Method of retirement (resurfaced, reconstructed, abandoned, or transferred) for mileage of each surface type retired each year (24 States and Puerto Rico).

Data for Mississippi were not available for the summaries prepared in connection with item 3. Information for California is for Federal-aid routes numbered 1, 2, 3, 7, and 8 and for Montana, U. S. Route 10 only.

There are eight major surface types for which individual service-life analyses were made: soil-surfaced, gravel or stone, bituminous surface-treated, mixed bituminous, bituminous penetration, bituminous concrete, portland cement concrete, and brick or block.

Definitions of these surface types, used in all phases of the highway planning surveys in determining the general type classifications constructed in the individual States, are found in the appendix (pp. 23–24). Definitions of the four methods of retirement are also included in the appendix.

Average Life Defined

The average service life of a road surface is the average period of time after construction that the surface remains in service prior to being replaced, resurfaced, reconstructed, or otherwise taken out of service for any reason or by any method. Stated in another manner, it is that period of time after construction during which the only operations performed on the road surface are those of maintenance as practiced by the various States.

It is a recognized fact, however, that a significant amount of construction work is done with maintenance funds in many States. This is particularly true in the low and intermediate surface types. In recording the original data summarized in this study, an attempt was made in each State to segregate construction from maintenance in a uniform manner regardless of the particular accounting practices in effect in a given State. The classifications of construction and maintenance operations generally followed in the road life study are those included in the tentative draft of the report to the 1938 meeting of the Subcommittee on Uniform Accounting of the American Association of State Highway Officials.[4]

Analysis Procedures

Survivor curve analysis procedures employed are substantially the same as those discussed at some length in the 1941 article. Reference should be made to this earlier study for an explanation of the mechanics for computing average service life. In the 1941 study, a single analysis was made of combined data for all States while in the 1949 study and the present one, individual analyses were made for each State and the results combined by weighting. The two procedures will yield the same results. One of the advantages of analyzing the data by individual States is that results are retained for each State and are available for use in further State studies.

The average life data included in this article represent estimates based on actual experience. Over the years there have been changes

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[4] Copies of this tentative draft were transmitted to all State highway departments under date of June 2, 1938, by the Subcommittee on Uniform Accounting, American Association of State Highway Officials.

Table 1.—Mileages constructed and mileages remaining in service on January 1, 1953, for each surface type [1]

Construction-year period	Soil-surfaced		Gravel or stone		Bituminous surface-treated		Mixed bituminous		Bituminous penetration		Bituminous concrete		Portland cement concrete		Brick or block	
	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service	Miles constructed	Miles remaining in service
1920 and prior	488. 1	6. 2	10, 713. 6	11. 5	2, 864. 4	596. 0	64. 0	6. 9	533. 9	118. 5	814. 7	115. 5	2, 826. 1	489. 6	728. 5	87. 3
1921–25	1, 165. 9	5. 5	22, 493. 4	421. 5	3, 665. 1	896. 8	317. 7	43. 2	2, 454. 8	517. 8	1, 510. 6	295. 9	12, 169. 1	5, 119. 9	598. 1	59. 1
1926–30	712. 6	----------	24, 048. 3	1, 360. 7	8, 662. 1	2, 464. 1	6, 015. 9	908. 4	2, 251. 6	663. 2	2, 035. 9	444. 3	16, 909. 5	11, 425. 9	183. 0	46. 5
1931–35	1, 307. 3	33. 2	17, 912. 8	1, 433. 3	11, 056. 7	2, 982. 0	20, 540. 3	6, 450. 0	3, 366. 9	1, 864. 2	2, 011. 5	790. 5	12, 938. 0	10, 691. 3	183. 8	54. 9
1936–40	1, 325. 4	52. 9	10, 136. 2	804. 9	12, 945. 1	5, 695. 9	17, 157. 0	9, 498. 1	2, 726. 2	2, 142. 9	4, 256. 9	2, 574. 7	6, 962. 5	6, 114. 5	49. 6	27. 5
1941–45	448. 0	18. 1	4, 688. 4	730. 8	7, 682. 7	4, 616. 9	7, 917. 5	5, 129. 1	1, 967. 6	1, 516. 5	4, 543. 6	3, 537. 0	2, 489. 5	2, 299. 5	14. 5	7. 9
1946–50	276. 0	25. 6	5, 716. 8	1, 537. 8	10, 623. 5	9, 072. 5	13, 882. 1	12, 203. 4	959. 6	815. 3	10, 081. 8	9, 685. 3	2, 982. 0	2, 963. 4	6. 1	1. 7
1951–52	74. 4	45. 7	1, 940. 5	1, 375. 9	4, 193. 9	4, 078. 6	6, 394. 8	6, 302. 1	300. 9	296. 4	6, 639. 0	6, 622. 9	1, 180. 0	1, 168. 1	1. 4	1. 4
Total	5, 797. 7	187. 2	97, 650. 0	7, 676. 4	61, 693. 5	30, 402. 8	72, 289. 3	40, 541. 2	14, 561. 5	7, 934. 8	31, 894. 0	24, 066. 1	58, 456. 7	40, 272. 2	1, 765. 0	286. 3

[1] Compiled from data submitted by 25 States and Puerto Rico for rural State or Federal-aid primary systems.

in construction methods and design standards. There have been periods of accelerated activity and periods when little or no construction was accomplished. Some roads have been kept in service too long, while others have been rebuilt before the end of their useful life. Maintenance has frequently been inadequate. There have been instances of over- and underdesigning. Throughout the past 35 years, nevertheless, there have been sustained improvements in the standards of highway design, construction, maintenance, and administration. Each of these factors has its influence upon service life, but individual effects cannot be evaluated with certainty. As a result of improvements which are continuously being made in design standards, for example, such factors as excessive grades, sharp curves, narrow roadway widths, and restricted sight distances formerly contributing to early obsolescence or structural failure are gradually being reduced to a minimum, or even eliminated.

The large backlog of needed replacements of highway facilities, which had accumulated during World War II, was mentioned in the 1949 study. An adjustment was made in analysis procedures on the assumption that there would be a somewhat higher than average rate of retirement for about 10 years after the war. Actual experience shows that for the first 7 years of the 10-year period, replacement rates have not been quite as high as predicted. However, State highway needs studies which are underway or have been completed show that deficiencies exist in amounts sufficient to warrant continuation of this assumption. In fact, if the accumulated deficiencies in the highway plant are to be overcome at the rates recommended in some of these long-range studies, it is likely that the probable remaining service lives may, in some instances, prove to be somwehat less than indicated by the data presented in this article.

Factors Influencing Results

In actual practice only a small percentage of road sections have a life exactly equal to the average. Thus, in estimating service life for a particular road section, it is necessary to consider such factors as age, structural condition, design features, location, and traffic usage which reflect conditions peculiar to that section. Only by the exercise of expert engineering judgment in the evaluation of these factors is it possible to arrive at an estimate of the remaining service life for a particular road section.

There are many factors that have impact on construction practice and in turn influence the trend of age of road surfaces and of expected service. These include administrative policy, availability of materials and manpower, change or influence of politics, increased activity due to State legislative action, construction activity in neighboring States and in other fields of construction, civil defense activity, and any unusual or extended nationwide highway program.

Limitations in a study such as this are understandable as data were submitted by 26 different reporting units. Each State has slightly different practices with respect to constructing and maintaining roads and in reporting data. These factors, in addition to those mentioned in the preceding paragraph, tend to have their influence on service lives, age and expectancy, methods of retirement, and so forth. Even with such limitations the findings are useful. Comparisons can be made, and the range of speculation on trends or unusual changes with respect to road-surface lives can be narrowed.

Mileage In Service

Table 1 shows for each surface type, by construction periods, the mileage constructed during each period, and the mileage remaining in service on January 1, 1953. Approximately 42 percent of the surfaced mileage on the primary rural State highway systems of the United States is represented. The proportions of each surface type included in this study are as follows:

	Percent
Soil-surfaced	5
Gravel or stone	20
Bituminous surface-treated	36
Mixed bituminous	43
Bituminous penetration	37
Bituminous concrete	50
Portland cement concrete	58
Brick or block	48
Average, all types	42

There are some mileages, particularly of the lower-type surfaces, for which the dates of retirement are known but initial construction dates are not available. This results primarily from the difficulty in locating records of early construction. Partial data in those cases are not included in the analysis.

The probable average service lives for each surface type by construction periods are shown in table 2. Estimates of average lives are given in this table for all constructed mileages reported. Because of the smaller mileages involved, the retirement trends for earlier construction are frequently more erratic than the trends for the larger mileages of more recent construction. The average life estimates for this earlier construction are more reliable, however, because of the greater experience.

Table 2.—Weighted probable average service lives for various construction-year periods for each surface type [1]

Construction-year period	Soil-surfaced	Gravel or stone	Bituminous surface-treated	Mixed bituminous	Bituminous penetration	Bituminous concrete	Portland cement concrete	Brick or block
	Years	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*
1905 and prior	- -	15. 6	34. 1	------	26. 4	23. 5	------	40. 0
1906–10	10. 5	9. 2	38. 8	------	24. 7	27. 8	41. 3	25. 1
1911–15	5. 6	15. 3	19. 5	16. 4	24. 7	21. 8	18. 4	21. 0
1916–20	12. 7	11. 5	19. 0	18. 4	17. 9	19. 8	23. 3	20. 7
1921–25	9. 1	10. 0	19. 1	14. 2	19. 0	20. 3	27. 0	19. 6
1926–30	6. 0	8. 8	16. 1	12. 5	18. 5	17. 8	26. 8	20. 5
1931–35	4. 0	7. 5	11. 7	13. 3	18. 8	16. 4	25. 7	16. 8
1936–40	3. 2	5. 6	12. 0	14. 7	19. 6	16. 1	23. 1	15. 6
1941–45	2. 3	5. 6	11. 1	12. 2	15. 4	*14. 1*	*21. 1*	*10. 5*
1946–50	1. 5	3. 1	9. 5	*11. 7*	*12. 0*	*16. 8*	*24. 0*	- ---
1951–52	2. 1	2. 8	*9. 3*	*13. 0*	*14. 7*	*17. 5*	*24. 3*	---
Average	5. 2	8. 3	12. 6	13. 1	18. 0	16. 8	25. 5	19. 9

[1] Based on analysis of data submitted by 25 States and Puerto Rico for rural State or Federal-aid primary systems. Average lives are to the nearest 0.1 year, but they should not be presumed accurate to this extent. The averages would be affected by excluding certain States or by including additional States. The entries in italics represent composite averages developed from the projection of trends for each State. All other entries represent averages obtained from analyses of construction and retirements where the mileages involved were sufficient to give a supportable estimate.

Table 3.—Comparison of average lives presented in the 1941 and 1949 studies with those listed in the present study

Surface type	Comparison period [1]	1941 study (26 States)		1949 study (16 States)		Present study (25 States and Puerto Rico)	
		Miles constructed	Average life	Miles constructed	Average life	Miles constructed	Average life
		Miles	*Years*	*Miles*	*Years*	*Miles*	*Years*
Soil-surfaced	1931–35	[2] 2, 542	[2] 5. 4	668	4. 5	1, 307	4. 0
Gravel or stone	1931–35	22, 793	6. 0	18, 999	5. 9	17, 913	7. 5
Bituminous surface-treated	1931–35	[2] 10, 286	[2] 11. 4	9, 301	[3] 7. 4	11, 056	11. 7
Mixed bituminous	1926–30	5, 610	14. 3	5, 801	12. 3	6, 016	12. 5
Bituminous penetration	1926–30	3, 725	17. 0	1, 851	16. 0	2, 252	18. 5
Bituminous concrete	1921–25	2, 362	17. 9	822	18. 1	1, 511	20. 3
Portland cement concrete	1921–25	6, 737	24. 4	8, 855	26. 1	12, 169	27. 0
Brick or block	1921–25	980	18. 2	331	20. 2	598	19. 6

[1] The most recent period for which data were published in the 1941 study.
[2] 1931–34.
[3] Wisconsin data had a great influence on the average life for miles constructed in the 16 States. If Wisconsin data are excluded, the average life is 10.2 years.

Comparison of Three Studies

There are certain differences in the average service lives presented in the 1941 article, the 1949 article, and the present one. Table 3 shows a comparison of the average lives for various surface types for the most recent 5-year periods for which data were listed in the 1941 study. No particular significance should be attributed to the differences in average lives shown in table 3. Variations in average service lives of this magnitude are not uncommon when the analyses are based upon different groupings and numbers of States. Furthermore, unusual construction practices in one State may have considerable effect upon the average for a group of States.

As previously mentioned, there are nine States included in the present study that were likewise included in the 1941 and 1949 studies. Based upon average lives developed in these two earlier studies, the 1941 article shows that of the mileage of all types in service on January 1, 1937, only 5 percent would probably still be in service on January 1, 1953; the actual amount was 8 percent. Similarly the 1949 article shows that of the mileage of all road types in service on January 1, 1946, 32 percent would probably still be in service on January 1, 1953; the actual amount was 34 percent.

There are, of course, greater variations within the individual surface types, but in total the results of these forecasts are sufficiently accurate to warrant considerable confidence in the analysis procedures and the average lives developed. Any differences from actuality can be readily explained. In the case of the 1941 study, the influence of the World War II period could not be anticipated; and in the case of both the 1941 and 1949 studies, there was no expectation that many surfaces would be kept in service beyond their normal life even though subjected, in most cases, to greater wear than in any previous period. In the 1949 study, it was assumed that the backlog of deferred work during the war would be overcome and normal trend resumed within a maximum of 10 years after the war, and corresponding adjustments of the service life computations were made. This assumption is proving correct although a somewhat longer period than 10 years for the "catch-up" should have been used.

Classification by Retirement Method

In all but one of the States included in the average life analysis, the retired mileages for various years were reported and classified in accordance with the method by which the retirement was made. The total mileage involved was 183,976.

The methods of retirement into which these mileages were classified are as follows: resurfaced, reconstructed, abandoned, and transferred. These retirement classifications are general in character and should be so interpreted. Definitions are included in the appendix.

Reversion in type (a surface reverts to a lower type through lack of adequate maintenance) also represents a distinct method of retirement. No retirements of this classification were reported in these data.

In table 4 are shown the total mileages retired and the percentage distribution by methods of retirement. With the exception of soil-surfaced, and brick or block types more than one-half of all the retirements hav been by resurfacing.

Table 5 shows the percentage distributio of miles retired by various methods for 5-yea periods (except for the earliest and mos recent years). Since the war there has been downward trend in resurfacing and an upward trend in reconstruction.

Analysis by Major Type Groups

Figures 2–5 show some of the results of th analyses presented in the preceding discussion For purposes of simplifying these charts, th surface types have been combined into thre major groups as follows:

Low type.—Includes soil-surfaced and grave or stone roads.

Intermediate type.—Includes bituminous surface-treated, mixed bituminous (thickness o surface and base less than 7 inches), an bituminous penetration (thickness of surfac and base less than 7 inches) roads.

High type.—Includes mixed bituminou (thickness of surface and base, 7 inches o more), bituminous penetration (thickness o surface and base, 7 inches or more), bituminous concrete, portland cement concrete, an brick or block roads.

Accumulated mileages constructed an remaining in service on January 1, 1953, ar shown by these groupings in figure 2. Figure 2–5 show clearly that construction program must necessarily continue after mileage ha been initially improved.

Estimates of service lives based upon actua retirement experience are shown in table 2 In general, the average lives for the mor recent years were assumed to remain abou the same or to increase slightly in relation t

Table 4.—Retired mileages for each surface type, and percentage distribution according to method of retirement (total for 1952 and prior) [1]

Surface type	Total retired	Methods of retirement				
		Resurfaced	Reconstructed	Abandoned	Transferred	Total
	Miles	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Soil-surfaced	5, 644. 1	37. 5	58. 1	1. 2	3. 2	100. 0
Gravel or stone	84, 051. 5	58. 0	30. 4	2. 5	9. 1	100. 0
Bituminous surface-treated	30, 518. 0	53 3	36. 7	2. 6	7. 4	100. 0
Mixed bituminous	30, 821. 8	57. 1	29. 8	3. 8	9. 3	100. 0
Bituminous penetration	6, 559. 0	51. 8	31. 4	3. 6	13. 2	100. 0
Bituminous concrete	7, 485. 8	60. 9	26. 0	2. 4	10. 7	100. 0
Portland cement concrete	17, 427. 3	66. 4	21. 3	2. 1	10. 2	100. 0
Brick or block	1, 468. 1	45. 3	44. 1	1. 3	9. 3	100. 0
Total	183, 975. 6	57. 0	31. 3	2. 7	9. 0	100. 0

[1] Computed from data submitted by 24 States and Puerto Rico for rural State or Federal-aid primary systems.

Table 5.—Percentage retired by various methods for all surface types combined, for various periods [1]

Method of retirement	Retirement period								
	1920 and prior	1921–25	1926–30	1931–35	1936–40	1941–45	1946–50	1951–52	Total, 1952 and prior
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Resurfaced	63. 5	45. 3	53. 0	59. 7	55. 5	59. 5	58. 6	54. 1	57. 0
Reconstructed	35. 7	49. 0	34. 9	26. 6	30. 9	28. 8	31. 5	36. 4	31. 3
Abandoned	. 4	. 6	2. 3	3. 8	3. 4	2. 7	2. 0	1. 6	2. 7
Transferred	. 4	5. 1	9. 8	9. 9	10. 2	9. 0	7. 9	7. 9	9. 0
Total	100. 0	100. 0	100. 0	100. 0	100. 0	100. 0	100. 0	100. 0	100. 0

[1] Computed from data submitted by 24 States and Puerto Rico for rural State or Federal-aid primary systems.

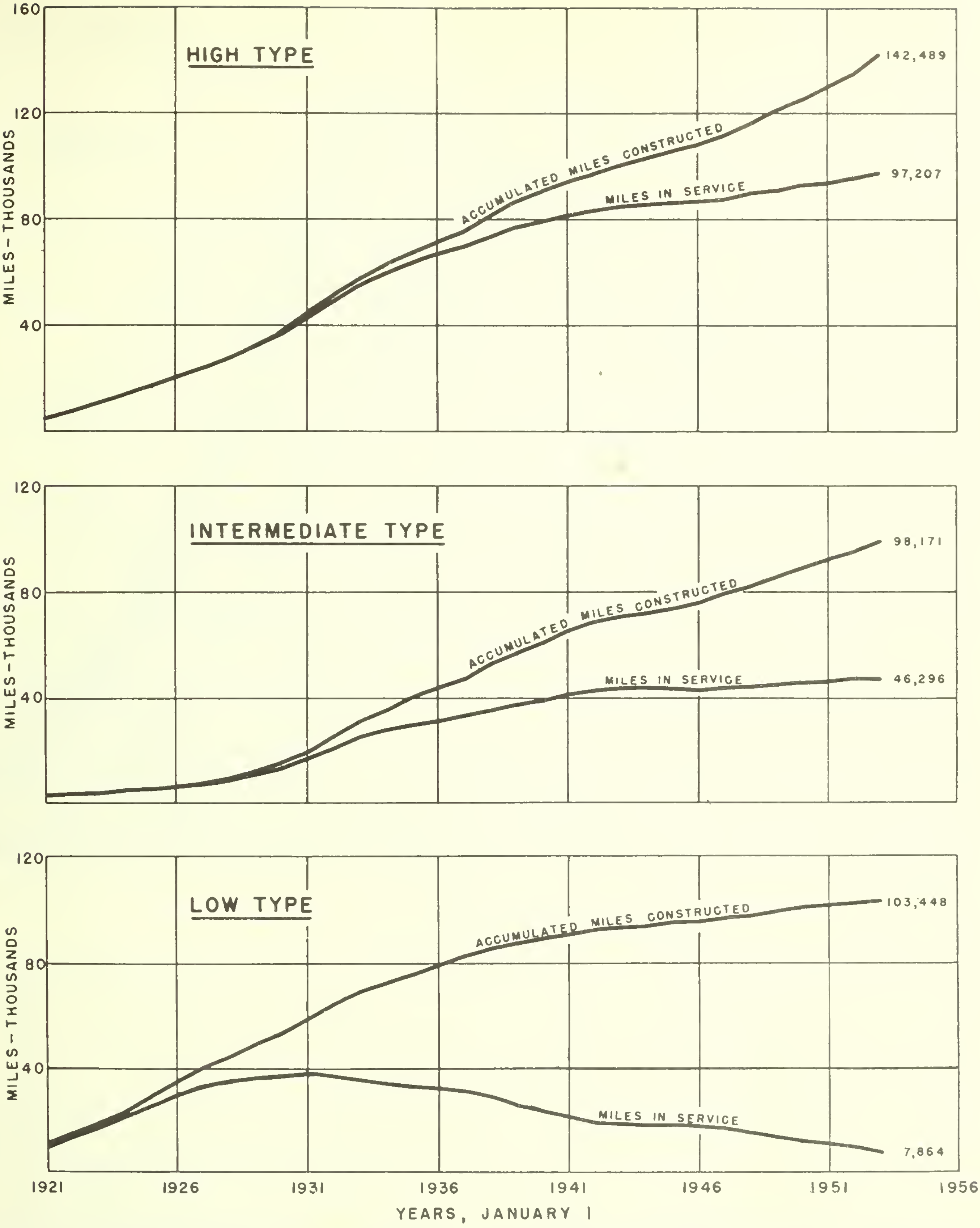

Figure 2.—Accumulated miles constructed and miles in service on rural State or Federal-aid primary systems in 25 States and Puerto Rico, according to low-, intermediate-, and high-type surfaces.

average lives for the most recent years having retirement experience. These assumptions will no doubt vary somewhat from the actual future experience. Since the bulk of the retirements within the next few years will come from the older construction, any minor differences from the assumed average lives for the more recent construction will not have any major effect upon the overall trends which are shown.

Trends in Service Life

Figures 3 and 4 show, for each 5-year construction period, the mileages of low, intermediate, and high types in service up to January 1, 1953, for the 25 States and Puerto Rico, and the probable rates at which these mileages will go out of service. Table 6 shows the total mileages in service on January 1, 1953, and the probable amounts which will still remain in service for 5, 10, 15, and 20 years in the future.

Table 2 shows the probable average lives for various construction-year periods. The

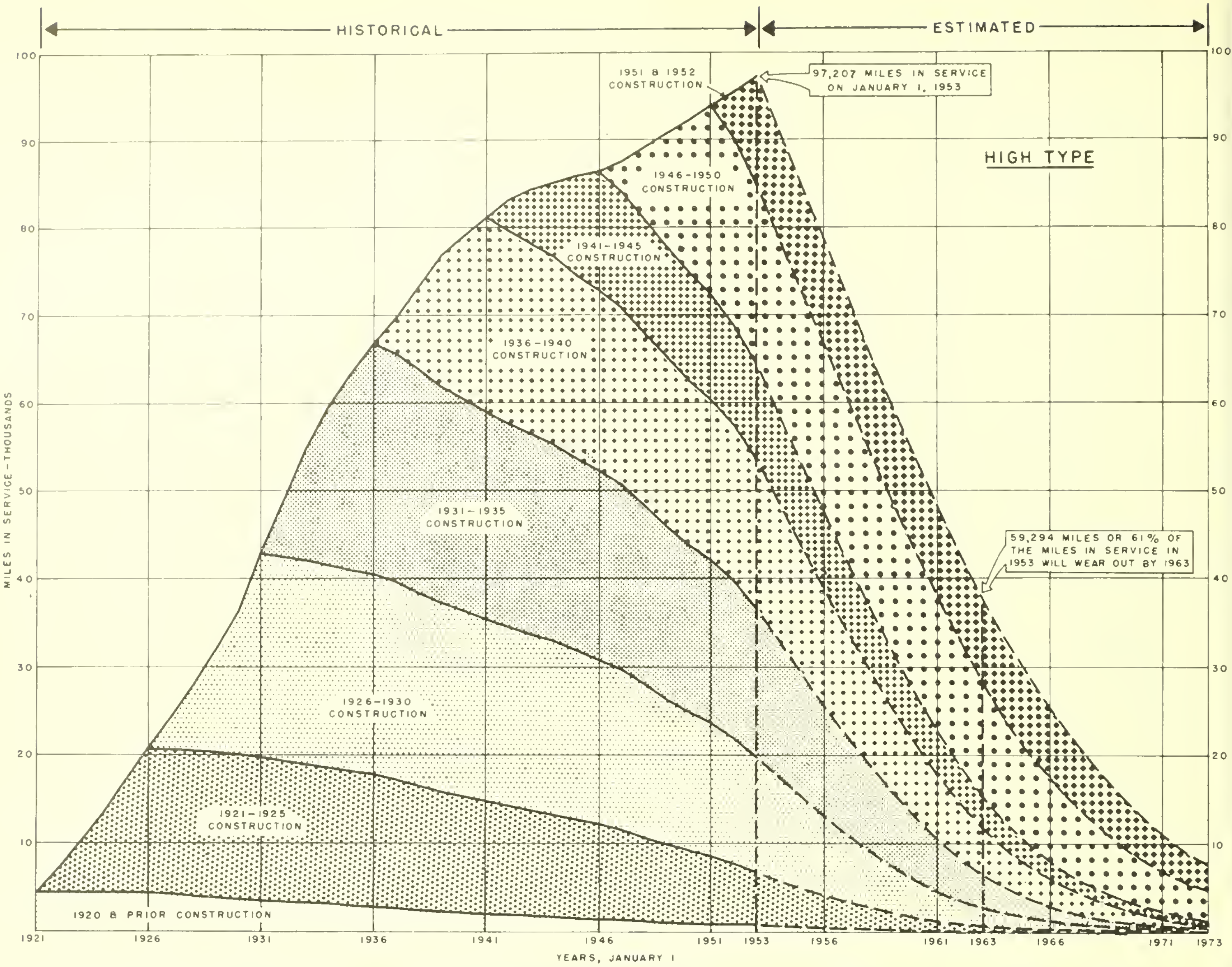

Figure 3.—Of the miles of high-type surfaces on rural State or Federal-aid primary systems in 25 States and Puerto Rico as of 1953, 61 percent will wear out by 1963.

average lives are the expectancies at the time of construction or at age zero. As the road system develops and becomes older, the average age of the surfaces increases and the remaining life expectancy becomes less. Also, as the system becomes older, mileages of earlier retirements are taken out of service; thus leaving in service a preponderance of those mileages with service lives exceeding the average life of the total original construction.

Life Expectancy

The probable life of the mileage in service is equivalent to the age plus the expectancy. Under certain conditions it is possible for the average age of mileage in service to exceed the average lives shown in table 2. This is true for low-type surfaced mileage, from which the miles in service have gradually been diminishing since 1931 (see fig. 4). Low types, when retired, tend to be replaced by intermediate or high types. Thus, there has not been sufficient construction of new low types to keep the average age of all low types in service from increasing from year to year. As a result, the low types now in service are quite old and have a short expectancy.

Table 6.—Mileage in service on January 1, 1953, and estimated percentages which will remain in various future years [1]

Surface type	In service on Jan. 1, 1953	Remaining in service on—			
		Jan. 1, 1958	Jan. 1, 1963	Jan. 1, 1968	Jan. 1, 1973
	Miles	*Percent*	*Percent*	*Percent*	*Percent*
Low	7,863.6	16.3	3.6	0.3	(2)
Intermediate	46,295.9	48.6	17.0	4.4	0.9
High	97,207.5	67.5	39.0	19.1	8.0
Total	151,367.0	59.0	30.4	13.7	5.4

[1] Based on analyses of data submitted by 25 States and Puerto Rico for rural State or Federal-aid primary systems.
[2] Less than 0.05 percent.

Table 7 shows the average age, remaining life expectancy, and total probable life of the mileages of low, intermediate, and high types in service, at 5-year intervals from January 1, 1921, to January 1, 1951; and a 2-year interval from January 1, 1951, to January 1, 1953. This information is also presented graphically in figure 5, which shows the effect of increasing age and decreasing expectancy of mileages in service.

The age of the three major type groups, as shown in table 7, has been increasing from 1921 to 1951. Similarly, the expectancy has been decreasing. Since 1951, there has been a leveling off which suggests that a period of stability has been reached. One means by which a more favorable condition with respect to expectancy can be obtained is by replacing worn-

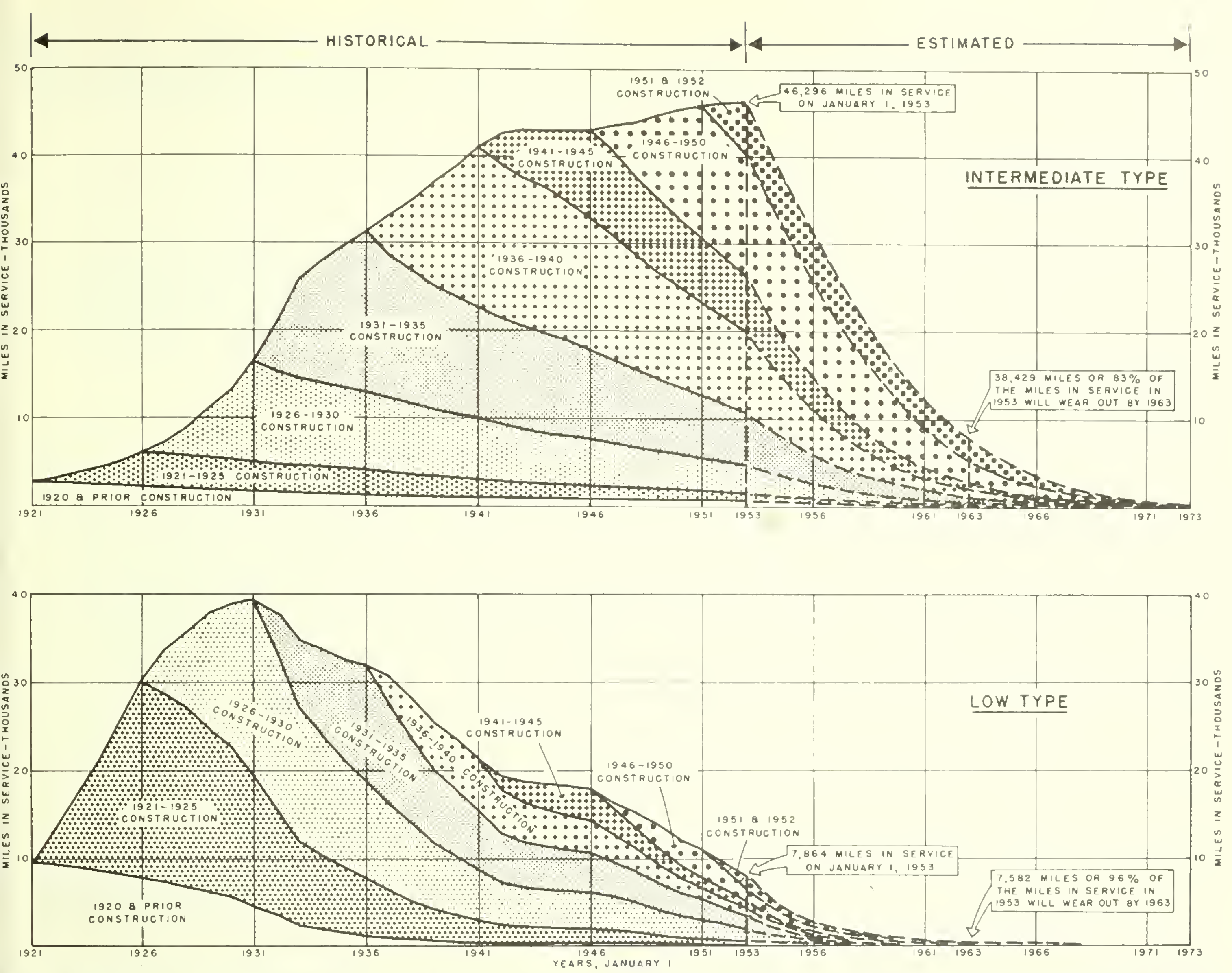

Figure 4.—Of the miles of intermediate- and low-type surfaces on rural State or Federal-aid primary systems in 25 States and Puerto Rico as of 1953, 83 percent of the intermediate type and 96 percent of the low type will wear out by 1963.

out roads with new ones having a longer service life. This, of course, means greater construction expenditures to build the new roads to the higher standards.

Road Life Studies Essential

The data presented relate only to the mileage of road surfaces constructed and retired. Road life studies also embrace research in construction costs, maintenance costs, and salvage values for elements of the highway, including grading and structures. Knowledge on these subjects will be extended as additional States bring their basic studies up to date and as these studies are continued and extended. The objective of efficient and economical management of the highway program is to provide facilities at such locations and to such standards that they can absorb the inevitable and continuing changes in traffic requirements with the least effect upon the ability of the highway plant to provide the maximum service at minimum cost. Data obtained periodically from road life studies are among the essential facts that contribute to the attainment of this objective.

Table 7.—Average age, life expectancy, and probable life of mileages in service at 5-year intervals to 1951 and 1-year intervals thereafter [1]

January 1,	Low-type surfaces			Intermediate-type surfaces			High-type surfaces		
	Age	Expectancy	Probable life	Age	Expectancy	Probable life	Age	Expectancy	Probable life
	Years	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*
1921	3.6	9.6	13.2	4.7	16.8	21.5	3.5	18.7	22.2
1926	3.8	7.9	11.7	4.7	16.8	21.5	3.5	21.3	24.8
1931	5.3	6.7	12.0	4.2	14.5	18.7	4.9	19.9	24.8
1936	6.6	7.2	13.8	5.5	12.3	17.8	7.0	17.0	24.0
1941	8.7	7.3	16.0	7.0	11.2	18.2	9.3	14.7	24.0
1946	11.7	5.0	16.7	9.7	8.5	18.2	12.3	11.4	23.7
1951	12.9	3.3	16.2	10.6	6.6	17.2	13.4	9.5	22.9
1952	13.1	2.9	16.0	10.8	6.1	16.9	13.6	9.2	22.8
1953	13.0	2.9	15.9	10.9	5.9	16.8	13.3	9.3	22.6

[1] Based on analyses of data submitted by 25 States and Puerto Rico for rural State or Federal-aid primary systems.

APPENDIX

Surface Type Definitions

Soil-surfaced road.—A road of natural soil, the surface of which has been improved to provide more adequate traffic service by the addition of (1) a course of mixed soil having A–1 or A–2 characteristics, such as sand-clay, soft shale or topsoil, or (2) an admixture such as bituminous material, portland cement, calcium chloride, sodium chloride, or fine granular material (sand or similar material).

Gravel or stone road.—A road the surface of which consists of gravel, broken stone, slag, chert, caliche, iron ore, shale, chat, disintegrated rock or granite, or other similar fragmental material (coarser than sand) with or without sand-clay, bituminous, chemical, or portland cement stabilizing admixture or light penetrations of oil or chemical to serve as a dust palliative.

Bituminous surface treated road.—An earth road, a soil-surfaced road, or a gravel or stone road to which has been added by any process a bituminous surface course, with or without a seal coat, the total compacted thickness of which is less than 1 inch. Seal coats include those known as chip seals, drag seals, plant-mix seals, and rock-asphalt seals.

Mixed bituminous road.—A road the surface course of which is 1 inch or more in compacted thickness composed of gravel, stone, sand, or similar material, mixed with bituminous material under partial control as to grading and proportions.

Bituminous penetration road.—A road the surface course of which is 1 inch or more in compacted thickness composed of gravel, stone, sand, or similar material bound with bituminous material introduced by downward or upward penetration.

Bituminous concrete, sheet asphalt, or rock asphalt road.—A road on which has been constructed a surface course 1 inch or more in compacted thickness consisting of bituminous concrete or sheet asphalt, prepared in accordance with precise specifications controlling gradation, proportions, and consistency of composition, or of rock asphalt. The surface course may consist of combinations of two or more layers, such as a bottom and a top course or a binder and a wearing course.

Portland cement concrete road.—A road consisting of portland cement concrete with or without a bituminous wearing surface less than 1 inch in compacted thickness.

Brick[5] *or block road.*—A road consisting of paving brick, stone block, wood block, asphalt block, or other form of block, with or without a bituminous wearing surface less than 1 inch in compacted thickness.

Methods of Retirement

Resurfacing.—Roads which are resurfaced or used as a base for the replacement type are so classified when the old surface is utilized more or less intact (with the exception of necessary scarifying, reshaping, or partial reworking of the surface) in the new construction which retires the old surface. Examples of this method are the retirement of a soil-surfaced road by surface treating, or the retirement of a gravel or stone road by utilizing it as a base or foundation for a mixed bituminous road or a bituminous penetration road. For surfaces which are retired by this method, it is obvious that the new or replacement construction must necessarily be along the same alinement and practically the same grade.

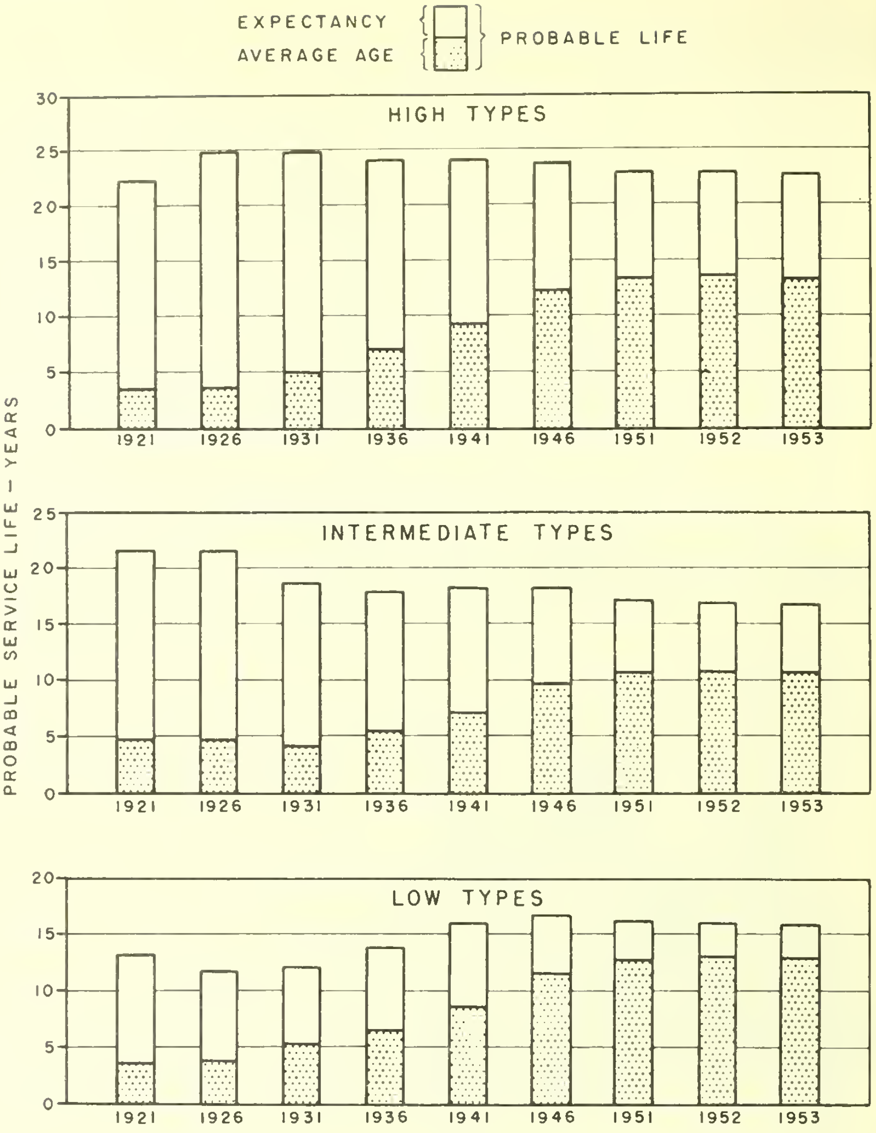

Figure 5.—Trends in average age, expectancy, and probable life of surfaced mileage on rural State or Federal-aid primary systems in 25 States and Puerto Rico for 5-year intervals 1921–51, and for 1952 and 1953.

Reconstruction.—When surfaces are retired by reconstruction, there is little or no salvage of the old surface and base. This classification includes old surfaces and bases that are torn up and not reused. Usually for types that are retired by this method, the replacement type is built along the same general alinement (generally within the limits of the existing right-of-way) with only minor improvements in horizontal curvature. Substantial improvements are usually made with respect to grades, however.

Abandonment.—When the new construction is on new location, the old road is classified as abandoned when it is no longer maintained or kept in service at public expense. The abandoned road may revert to a private road, be barricaded to public travel, or be removed. Sometimes because of changes in land usage, such as abandonment of factories, and removal or construction of railroad facilities, roads may be abandoned without involving new construction.

Transfer.—A retirement by transfer is similar to an abandonment except that the old road is continued in service after being dropped from the State or Federal-aid system and is maintained by county or other authority responsible for the upkeep of roads not on the State or Federal-aid system. A transfer is not a retirement in the sense that the road has rendered its total service to the public, but merely that it has rendered its complete service as a primary State or Federal-aid highway. Retirements by transfer are generally the result of functional obsolescence involving alinements and grades which are unsatisfactory for existing traffic conditions. A new road is built on new alinement and improved grades, and the old road remains in service usually because of the necessity of providing for local traffic usage. After the new road is placed in service on the State or Federal-aid highway system, the State will no longer wish to maintain the old road, and the county or other local authority generally takes over this responsibility. If the road is no longer used, it is considered an abandonment.

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[5] Vitrified paving-brick roads were reported by the States separately from other types of brick or block roads. Because of the small mileages involved, these two types are combined in this article. Approximately 99 percent of the construction of these two types included in the study is vitrified paving brick.

The Investment Analysis Approach to Estimating Highway Needs

BY THE FINANCIAL AND ADMINISTRATIVE RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported [1] by FRED B. FARRELL
Chief, Highway Cost Section

Research on the growth and depreciation of the investment in highways is relatively new. The basic data for such research are obtained from the road-life study phase of the Statewide highway planning surveys. Some of the initial applications of the findings are very promising. One of these, as discussed in this article, involves the estimating of highway needs.

The end product of a highway needs study is, of course, the formulation of a financial plan to meet such needs. This inevitably requires consideration of many financing alternatives, and in the case of credit financing proposals, requires a year-by-year listing of capital outlays required to reach adequacy within a given time period and thereafter to sustain adequacy. Not only does the investment analysis approach provide a quick means of estimating total needs, but it also provides a method of scheduling such needs on an annual basis over extended future periods.

THE time and effort spent by highway departments on highway needs studies over the past few years have paid dividends. The magnitude of the highway problem, locally and nationally, has been brought into proper perspective. The engineer's knowledge of geometric and structural needs has been translated into understandable dollars and cents terms that have become accepted by the general public.

Engineering Approach to Needs

The "engineering approach" is the usual means employed in estimating costs to improve road systems to a state of adequacy. In this approach, data on traffic, condition, deficiencies, and service characteristics are compiled for each individual road section. Engineering field checks and inspections are then made, needs determined, and costs of improvement estimated and identified either as an immediate need or as a future need within the next 5, 10, 15, or 20 years.

Under such an approach most of the needed improvements on existing roads fall in the category of "needed now." Successively lesser amounts show up as being needed each year thereafter. Since any actual improvement program to meet such a needs schedule would not be practical, it is the usual practice to take the results of the engineering approach and rework them in the office so as to produce a realistic program, say for 10 years This 10-year "catch-up" program would then include all the measured needs within the first 10 years *plus* an allowance for stop gaps and replacements.

Stop gaps represent construction necessary to keep roads in operation until the final improvement can be made. Replacements represent work (principally resurfacing) necessary in the last part of the 10-year period on roads built in the first part of the 10-year period. Some allowance must also be made for those improvements which are "needed now" but which are deferred to the fifth or sixth year of the 10-year catch-up program. When eventually built, the volume of traffic for which they will be designed will be somewhat higher. This could result in an increase in the original cost estimate.

The 10-year catch-up program represents only one condition. For purposes of financial planning, costs of 12-year, 15-year, 20-year, and other catch-up programs must also be prepared and studied. Additionally, it is desirable to develop future costs for the years following the catch-up period so as to enable long-range credit financing proposals to be worked out. In these particular cases, money is borrowed during the catch-up period and repaid afterwards. Obviously, a sound financing plan must take into account not only the debt repayment but also the construction and replacement needs in the years following the catch-up period.

In summary, the outstanding advantage of the engineering approach, especially for the principal road systems, is that it produces a clear cut, supportable cost estimate based upon study of the physical needs of individual road sections. Substantial amounts of time and effort are required, however, in making these estimates in the first instance. And although progress is being made in streamlining the engineering approach, there is at present no quick and easy means of rearranging, recasting, and spreading out basic needs study data so as to provide long-range, year-by-year, cost estimates for the many program alternatives that crop up in planning a financial program.

Investment Analysis Approach to Needs

The "investment analysis approach" offers considerable promise in this respect. This approach can be used in those States and on those road systems where the road life study is up to date. It is based upon the premise that under a condition of sustained traffic increase over a period of several years, there should be a corresponding increase in the investment (cost-new less depreciation) in the highway plant.

In other words, if traffic were to increase 40 percent over the next 10 years there would need to be a corresponding increase of about 40 percent in the investment in highways over these same 10 years. Of course, today's highways are not adequate. Hence, unless this backlog of existing deficiencies is also overcome, the 40-percent increase in investment over a 10-year period would not result in adequate highways. It would simply continue to keep the highways at the same relative position with respect to meeting traffic needs at the end of 10 years as they are today.

An illustration may clarify this point. Three years ago, the results of a nationwide study of capital investment in highways were reported at the annual meeting of the Highway Research Board.[2] Figure 1 is taken from this study. It shows three curves, all based on January 1, 1953, price levels. The top curve shows the accumulated capital outlays made for all roads and streets in the United States up to January 1, 1953. The middle curve shows the amounts remaining in service after making deductions for construction work that had been retired and is no longer in existence. It represents the original cost (new) of all roads and streets still remaining in service. But all of these roads and streets are not brand-new. They have aged and a portion of their original service life has been used up. In other words, they have depreciated. The depreciated amounts are represented by the bottom curve.

It may be well at this point to note that the basis for computing this chart was the yearly outlays for construction. The accumulation

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[1] This article was presented at the 35th Annual Meeting of the Highway Research Board, Wash., D. C., Jan. 1956.

[2] *The capital investment in highways.* by Fred B. Farrell and Henry R. Paterick. Proceedings of the Highway Research Board, vol. 32, 1953, pp. 1-11.

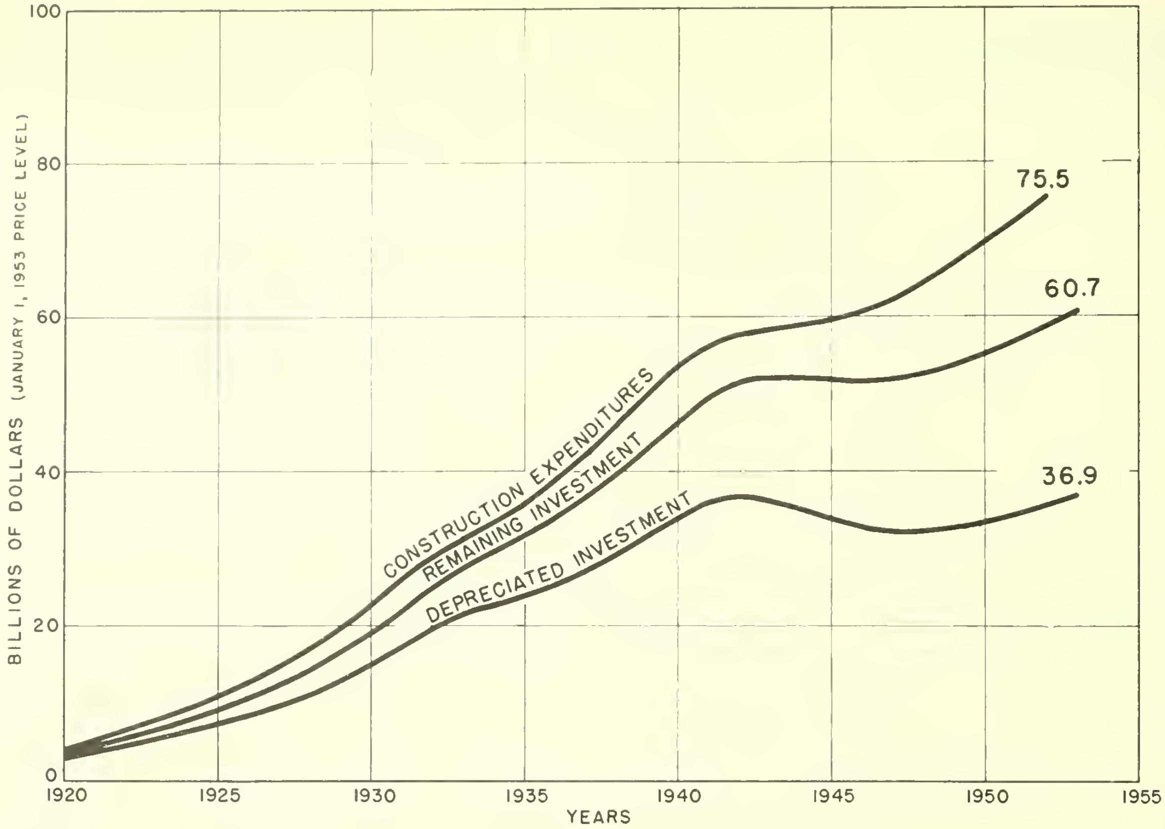

Figure 1.—Accumulated construction expenditures, remaining investment, and depreciated investment for all roads and streets in the United States up to January 1, 1953.

of these construction amounts is the top curve. From this top curve as a starting point the road-life study data and analysis procedures make it possible to compute the bottom curve—depreciated investment. Conversely, if the starting point had been the bottom curve—if all that was known was the trend in depreciated investment—it would be possible to reverse the computations to find out what the construction amounts are. This is called the "reverse computation," and the process is mentioned later.

In summary, figure 1 shows that up to January 1, 1953, (1) $75.5 billion had been spent for construction of all roads and streets in the United States, (2) $60.7 billion was the amount (cost-new) still in service, and (3) this total, when depreciated for service already consumed, amounted to $36.9 billion. It is the bottom curve, depreciated investment, that is primarily involved in the investment analysis approach. It is this trend that should correspond to the traffic trend.

The solid curve in figure 2 shows the trend in depreciated investment referenced to the years 1937–41 as a base of 100. This trend corresponds to the one for depreciated investment (bottom curve of fig. 1), with the exception that it includes certain minor adjustments due to price index revisions that were later considered desirable.

Selection of Base Period

The years 1937 to 1941 have been selected as a base of reference because they represent one of the most favorable periods in the development of highways. True, there were many highway needs during this period. But on the average it was a period during which highways reached their highest level of development in relation to the traffic demands imposed upon them. Prior to this period there was a sustained increase in the level of development. Subsequent to this period and after World War II, the traffic growth far outstripped growth of the highway plant.

Gap Between Investment and Traffic Widens

This can be readily seen by inspection of the traffic trend which is plotted as a dashed curve in figure 2. There is a similarity in the two trends up to 1941. The traffic then fell off until 1944, made a rapid recovery, and since 1946 has been increasing steadily. The trend in depreciated investment likewise dipped during the war. This was due to the fact that highways continued to depreciate during a period when capital outlay was drastically curtailed. After the war a slow recovery began—much slower than the traffic trend. Inspection of figure 2 shows just how slow it was. In fact it took 7 years of postwar construction for the depreciated investment to overcome the wartime dip and recover to the level of 1941.

The similarity of the two trends in the prewar years is sufficient to raise a question as to whether such similarity might not have continued in the postwar years under an adequate highway program. Certain rough computations made during 1953 seemed to support this hypothesis. Assuming that the hypothesis was correct, it would follow that if traffic could be predicted for the future, then th level of needed highway investment coul likewise be predicted. Then, by knowing th existing level of investment and the future leve of needed investment, it would be a simpl matter to compute the annual capital outlays necessary to raise the depreciated investment from one level to the next. This would be done by the "reverse computation" proces previously mentioned. It would give the construction needs.

Application of Reverse Computatio Process

An opportunity presented itself to test this theory early in 1954. At that time a highwa needs study using the engineering approach was being made in West Virginia, the results of which would not be available until mid-year. In the meantime, studies had been made in West Virginia of probable future travel trends. Also available were the results of the salvage value and investment analysis which were made as part of the road life study on the primary rural State highway system of West Virginia and which covered all construction and retirements up to 1954.

With this as a starting point, an analysis of the investment in the system was made which produced a series of values for the depreciated investment in terms of constant 1953 dollars for each year up to January 1, 1954. The resulting trend was then referenced to a base period (1937–41=100) and compared with the travel trend and its future extension. These two trend lines are shown in figure 3. There is a wide gap between the two trends in 1954.

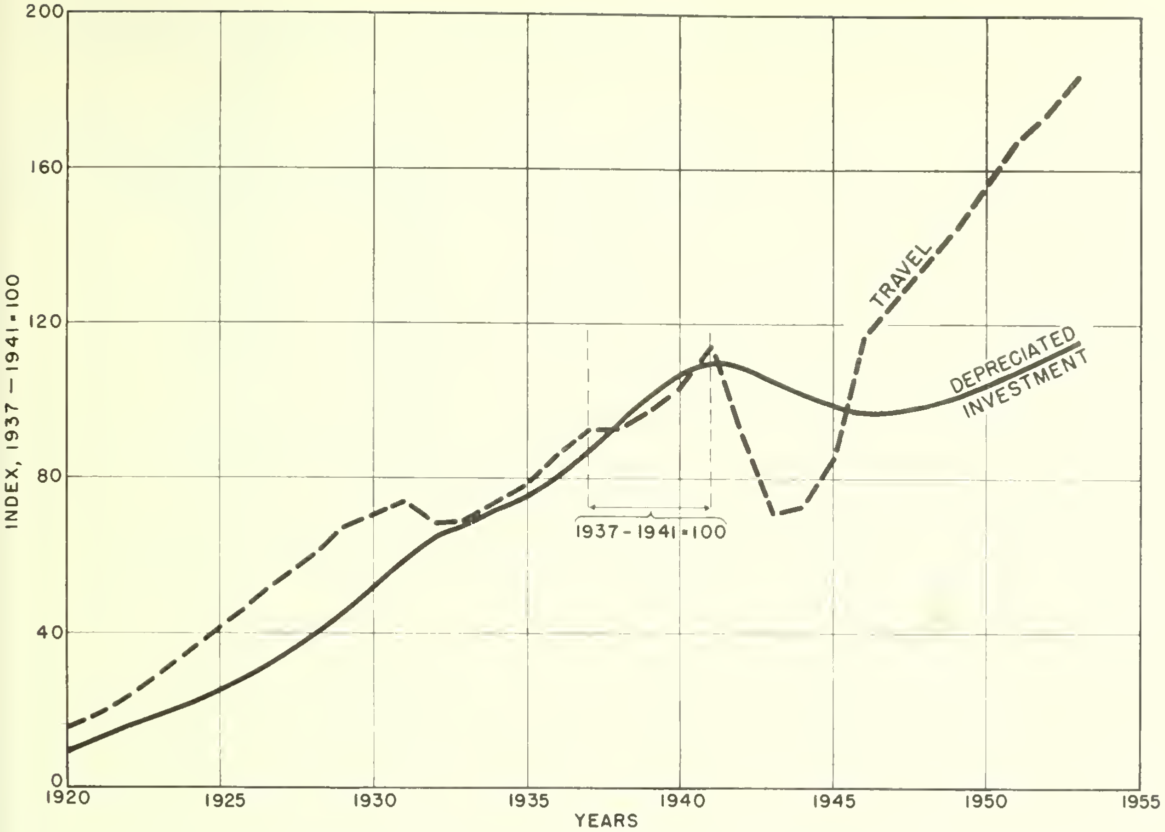

Figure 2.—Trends in travel and depreciated investment for all roads and streets in the United States up to January 1, 1953.

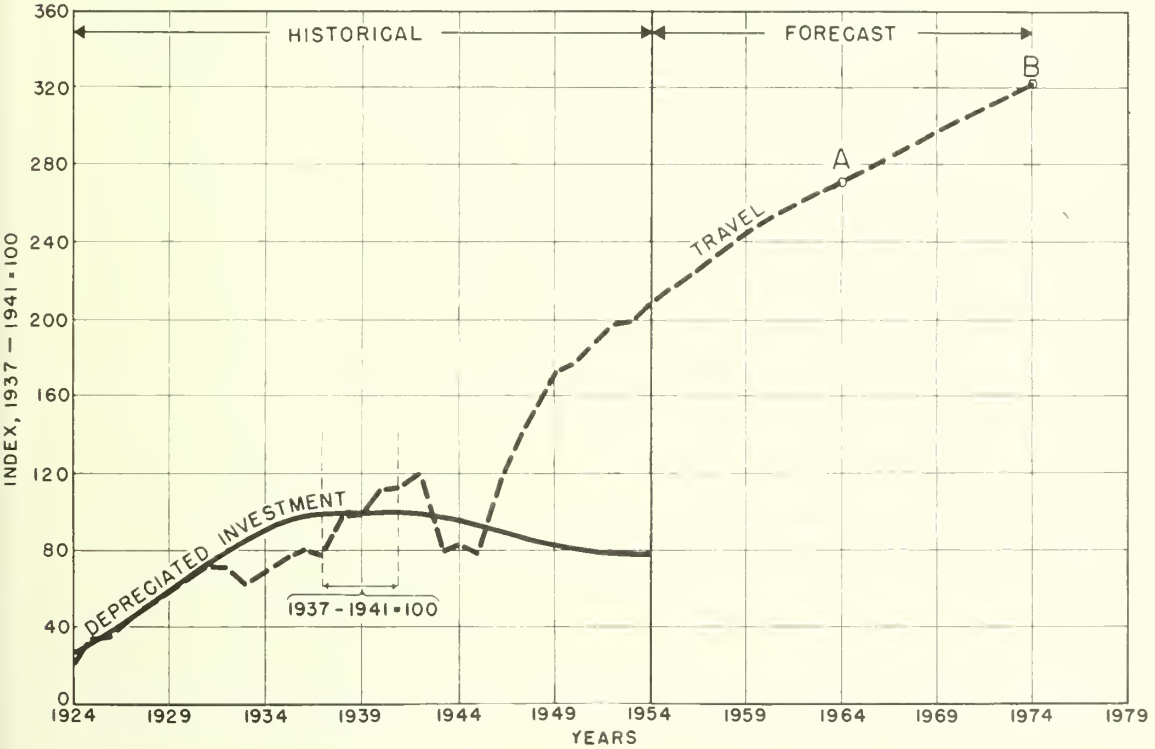

Figure 3.—Trends in travel and depreciated investment for the primary rural State highway system in West Virginia.

The next step was to test the theory that the trend in needed investment should basically be the same as the travel trend. To do this simply involved the assumption that for a 10-year program, for example, the trend in depreciated investment would catch up to the travel trend in 10 years (to point A by 1964). If a 20-year program was assumed, it would catch up in 20 years (to point B by 1974); and so on for any selected catch-up period. Once the trend in depreciated investment catches up to the travel trend, it is assumed that adequacy will be sustained thereafter. This means the investment trend then becomes identical to the travel trend.

Next, computations were made of the construction amounts necessary to raise the level of investment up to the travel trend to points A, B, and other selected catch-up program periods. This was done by the "reverse computation" process. To do this required certain assumptions as to the investment lives of recent and future construction of various roadway elements: grading; low-, intermediate-, and high-type surfacing; and structures. The decision as to the proper service lives to use in these cases is the most critical part of the analysis. However, it is probably no more critical than certain decisions that must be made when evaluating needs on a project-by-project basis in the engineering approach. Such estimates of service lives, of course, can be reasonably approximated from a study of past trends.

Allowances must also be made for the increases in service lives that will result when roads are rebuilt to the structural and geometric design standards called for in the needs studies. It is quite possible, for example, that the service life of future surfacing would be increased by as much as 25 percent over the present average. For grading on the highest type roads, future service lives may be as much as 75 to 100 percent greater.

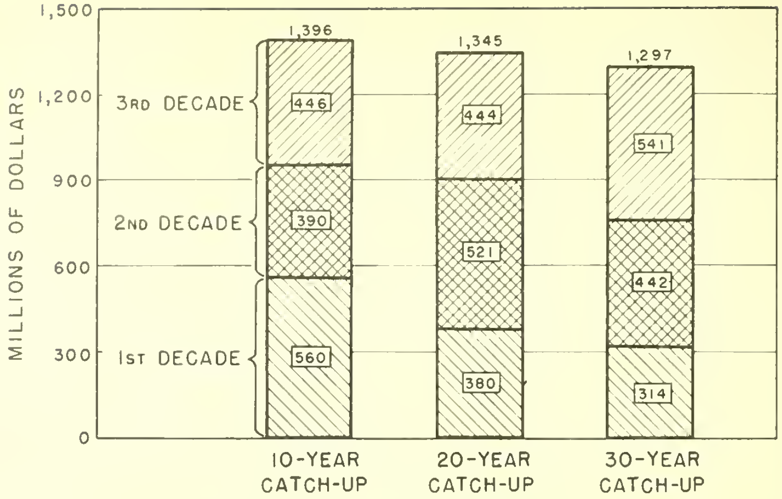

Figure 4.—Average construction needs in terms of 10-, 20-, and 30-year catch-up programs for the rural State primary systems in Missouri, Washington, and West Virginia.

Needs Computed for Three States

The cost estimates developed from the engineering approach in West Virginia became available in mid-1954. These estimates were then compared with the earlier ones of the construction needs computed in reverse by the investment analysis approach previously described. The agreement between the two estimates seemed to bear out the reasonableness of the latter approach. The following is quoted from pages 59–61 of the 1954 report "A Factual Study of Highway Needs in West Virginia[3]":

> A completely separate statistical analysis of capital investment, excluding right-of-way, in the rural State primary system was prepared as part of this study . . .
>
> Adjusted to 1953 price levels and reduced 10 percent to represent the proposed rural State Trunkline System, the analysis shows that:
>
> $38 million annually would be required for construction alone to meet traffic needs in 20 years. That compares with $35 million per year derived from the field studies . . .
>
> A 15-year program, for construction only, would require $47 million annually, compared to $45 million determined by field studies. . . .
>
> * * * * *
>
> . . . The field study results are less than, but similar to, the investment analysis, although developed on a wholly different basis. The similarity gives assurance that both methods are adequate . . .

Subsequently, similar analyses were made for the rural primary highway systems of Missouri and Washington. In each case reasonable approximations of total needs were obtained. Additional studies are now being made for other States where road life studies are sufficiently far advanced.

Research on the investment analysis approach is still in its initial stages. There is, however, one general finding that warrants mention. It has been found that over a 30-year period, the total capital outlay needed to build an adequate highway system and thereafter keep it adequate is about the same regardless of the time taken for the initial catch-up program.

This is shown in figure 4 for 10-, 20-, and 30-year catch-up programs. Data used are averages for the rural primary systems of Missouri, Washington, and West Virginia. The height of the bars represents the total construction costs over a 30-year period. The total costs are about the same in all three cases, the principal differences being in the relative amounts needed in the first and second decades of each catch-up program. In other words, the first bar shows that for a 10-year catch-up program, there would be a heavy expenditure in the first decade while catching up to adequacy. Thereafter, in the second and third decades the cost to sustain adequacy would be somewhat less. For the 30-year catch-up program, on the other hand, the costs would be the least in the first 10 years and successively greater in the next 2 decades. Total costs over 30 years would be about the same in each instance. But in one case, the road system would become adequate in 10 years and thereafter be kept adequate. In the other case, the road system would not become adequate until 30 years have passed.

Evaluation of Investment Analysis Approach

Perhaps the most attractive feature of the investment analysis approach to estimating highway needs is the ability to process the data in various ways. For example, in the study of alternative financing plans, it is desirable to have a year-by-year array of construction needs during various catch-up periods, say 10, 12, 15, and 20 years. It is also desirable to have a year-by-year array of future needs for 10, 20, or even 30 years after the shorter catch-up periods. Similarly, it may be desirable to know at what rate the highway needs will be overcome under various financing proposals.

The investment analysis approach can provide such data readily. Its outstanding advantages are the speed with which needs estimates can be prepared, the small amount of manpower required, and its extreme flexibility in arraying needs data in various ways. But there are also disadvantages in that it treats road systems as lump sums rather than by individual road sections.

Possibly the best circumstances exist in those cases where a State has *both* the results of an engineering study of needs and an up-to-date road life study. In such instances, the investment study can be used as a means of introducing flexibility into the engineering study results. As a case in point, a given State may have a well developed engineering study which provides an estimate of the cost of bringing each road section up to adequacy within 10 years. With such information as a base, the investment analysis approach can then be used to adjust the needs study findings to show costs of 12-, 15-, and 20-year, or any other catch-up period and to extend them into the future beyond the catch-up period.

Further, there are cases where a needs estimate has been prepared upon certain assumptions as to future traffic growth. If these assumptions were to be changed for one reason or another, it would obviously affect the needs estimate. The making of such revisions could pose considerable difficulties in the engineering approach. Under the investment analysis approach a revised estimate could be prepared quickly.

Additional investment studies and further refinement of the analysis processes are desirable before specific relations and findings can be presented which will adequately reflect the widely varying conditions encountered in the various States. The concept and technique of analysis are, however, quite simple. Preliminary results attest to the reasonableness of the investment analysis approach. They give indication that this product of the road life study can be effectively used in estimating highway needs for various conditions of future traffic and in arraying such needs on a year-by-year basis for financial planning purposes. The advantages of the investment analysis approach become doubly apparent in States where the road life study is up to date. In such cases, the staff and time requirements for the analysis are only a fraction of those needed for the engineering approach.

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[3] A report to the West Virginia Legislative Committee. Prepared under the terms of House Concurrent Resolution No. 4, adopted Feb. 5, 1953.

A Six-State Classification Study of Engineering Personnel

BY THE FINANCIAL AND ADMINISTRATIVE RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

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Because an earlier study revealed wide variations in the number of engineers employed by the several State highway departments per million dollars of capital outlay for highways, analyses were made in six States to determine the significance of these variations. The earlier study, made in 1954, indicated a range of 2.0 to 28.2 engineers per million dollars of capital outlay among all States.

It was realized, of course, that differences among the States in classifying and reporting engineering personnel might account to a considerable extent for the variations, but the possibility that the variations might be indicative of relative operating efficiency was also considered. In an effort to explain the variations, studies of engineering classification and related matters were conducted during the summer and fall of 1955 in Mississippi, Nebraska, Oregon, Vermont, Washington, and Wisconsin.

It was found that classification procedures and reporting methods definitely affect the results reported by the earlier study. Two of the six States studied, for example, had reported only registered engineers, while the other four States had reported all personnel classified as engineers by civil service or merit system provisions, regardless of professional qualifications.

The present six-State study reveals that of the 2,114 employees classified as engineers, only 31 percent were registered engineers, 35 percent were civil engineering graduates (about half were registered), and 52 percent were neither civil engineering graduates nor registered.

This article also relates the number of engineers employed to the number of subprofessional employees, to program characteristics, and to management practices and procedures. The findings are not conclusive, because of the many intangibles involved and the relatively few States included in the study. It is established, however, that by combining the number of engineers and engineering aids the extreme variations in the number of engineering employees per million dollars of capital outlay are reduced considerably.

A STUDY was undertaken by the Highway Research Board late in 1954 to determine the number of professional engineers employed by all State highway departments, the number of engineering positions it would be necessary to create in order to handle the work then being handled by consulting firms, and the number of additional engineers needed in order to work at the highest level of effectiveness.[2] The States reported a total of 18,034 engineers employed, consulting work equivalent to another 4,192 engineering positions, and a need for 3,990 additional engineers for fully effective work.

Subsequent analysis indicated that the number of engineers reported per million dollars of capital outlay varied from 2.0 to 28.2 in the individual States. These wide variations were somewhat of a surprise, and their possible significance prompted further study. It was realized, however, that differences among the States in classifying and reporting engineering personnel might account to a considerable extent for the variations.

There was also the possibility that the variations might be indicative of relative operating efficiency. If so, it seemed likely that those States with a low number of engineers per million dollars of capital outlay might furnish ideas for the utilization of engineering manpower which would be of value to other States. In any event, no conclusions could be reached without a much more detailed analysis.

Accordingly, it was decided to make a detailed study of engineering classification and related matters in six selected States—Mississippi, Nebraska, Oregon, Vermont, Washington, and Wisconsin. In selecting these States, consideration was given to the relative number of engineers per million dollars of work, geographic location, rural-urban characteristics, total amount of program, amount of work done by consultants, and number of additional engineers needed for fully effective work. Information on the number of engineers employed per million dollars of capital outlay for the six States selected is presented in table 1, on the basis of data reported in the 1954 study.

That study, in asking for information on the number of professional engineers employed, defined a professional engineer as a "registered professional engineer, or one qualified to register." Since this definition was subject to interpretation by the States, it was decided that one of the primary concerns of the new studies should be the professional qualifications of employees classified by the States as engineers. Also, it was decided to extend the studies to include engineering aids, as well as engineers, and to relate the number of engineers and engineering aids employed to both program characteristics and management practices and procedures. Such studies were conducted in each of the selected States during the summer and early fall of 1955.

Classification Plans of the Several States

Since these studies are concerned primarily with classification, it is desirable at this point to comment briefly on the classification plans of the States included in the studies. The highway departments of Oregon, Vermont, Washington, and Wisconsin all operate under formal civil service systems and have classification plans of the graded type; that is, Engineer I, II III, IV, V, and Engineering Aid I, II, III, or A, B, C, and so forth. The Nebraska Department of Roads and Irrigation has for years maintained an informal merit system for its technical employees, and also has a graded classification plan. In Mississippi, job titles are related to specific duties, such as junior engineer of final plans, senior instrumentman, junior draftsman, rodman, and others which are difficult to correlate with the several classes of a graded classification plan.

Moreover, even in the five States with graded classification plans, correlation is not a simple matter. There are several reasons for this. In the first place, a graded classification plan usually includes in addition to engineers and engineering aids, several miscellaneous classes such as draftsman, radio technician, laboratory technician, traffic recorder, and on occasion even laborer, which can be included in either the engineer or engineering-aid categories, and sometimes in both. Also, the duties performed by an Engineer I in some States are performed by high-grade engineering aids in other States. In most States a civil engineering graduate can be hired as an

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[1] This article was presented at the 35th Annual Meeting of the Highway Research Board, Wash., D. C., Jan. 1956.

[2] *Engineering-personnel needs for highway departments*, by M. Earl Campbell and L. R. Schureman. Highway Research Board, Bulletin 106, 1955.

Engineer I, but in others he must be hired as an engineering aid and cannot be classified as an engineer until certain service requirements have been met.

The matter of registration is a confusing one, too. Some States require that engineers in a particular classification or salary scale be registered, but others require registration only in connection with certain duties. Also, some States require registration for particular grades, while others require only eligibility for registration. In Wisconsin, for example, an Engineer IV must be eligible for registration, while an Engineer V or higher must be registered. In Oregon, classification as a Civil Engineer IV or higher requires registration. Registration at the associate engineer level is preferred in Washington and is required at the senior and higher levels. In Mississippi, only field engineers at the project level and higher are required to be registered.

All of this discussion indicates, of course, that there are wide differences in the qualifications of the engineers classified as such by the several States. It also suggests that there may be some variation among the States in the relation of classification to duties. Both are important matters in determining the number of engineers employed by the State highway departments, but because of time limitations it was impossible to conduct the interviews necessary to determine the relation of classification to duties. The number of civil engineering graduates and registered professional engineers included among the classified engineers in each State was tabulated, however.

Although the 1954 study reported engineers employed, those equivalent to consulting work, and additional engineers desired, the six-State study is concerned only with engineers employed, because of the intangibles involved in studying the other two categories. Also, while the 1954 study reported engineers by function to which assigned, such as construction, design, maintenance, and so forth, this functional distribution is not carried forward here because of the relatively few engineers assigned to other than a few major functions. In computing the number of engineers employed per million dollars of capital outlay, however, those assigned to maintenance are omitted in all cases, because their efforts do not affect the capital outlay accomplishment.

It should also be pointed out that in most cases the number of engineers employed in a particular State does not change much from year to year, nor from season to season. Neither does the number of engineering aids change much from year to year, although this number may change substantially at different seasons of the year. Program amounts in a particular State, however, may vary widely from year to year with a corresponding pronounced effect on the number of engineering employees per million dollars of capital outlay.

Table 1.—Engineers employed and number per $1 million of capital outlay in six selected States

State	Number of engineers [1] assigned to—			Capital outlay [2]	Engineers employed per $1 million of capital outlay [3]
	Maintenance	Other than maintenance	Total		
				Million dollars	
Mississippi	11	100	111	26.3	3.8
Nebraska	20	225	245	16.5	13.6
Oregon	26	426	452	40.1	10.6
Vermont	25	128	153	5.2	24.6
Washington	12	194	206	46.3	4.2
Wisconsin	34	388	422	33.4	11.6
Total	128	1,461	1,589	167.8	8.7

[1] As reported in Highway Research Board Bulletin 106, 1955, p. 4.
[2] *Highway statistics*, 1954, Bureau of Public Roads, table SF-4, p. 55.
[3] Excluding those assigned to maintenance.

Conclusions

Generally, the findings of the six-State study are relative rather than absolute or final. The fact that only six States are included is itself a limiting factor. For any particular State, the information reported here might have been widely different a year ago, or might change radically during the course of the next year. Whether the six States selected for study are representative of the other States is not known. Nevertheless, it is believed that the studies contribute materially to the overall engineering manpower problem, if only because of the questions they raise.

With respect to the wide variations in the number of engineering employees per million dollars of capital outlay in the individual States, as reported in the 1954 study, some of the variations have been explained satisfactorily, and some have not. In any event the complexity of the situation has been demonstrated. It appears that the variations are influenced by a number of factors, and that some of them lend themselves to analysis, while others do not. Whether or not the ratio of engineering employees to capital outlays is a valid indication of overall operating efficiency remains a moot question.

The really significant finding, however, is that no one really knows how many engineers or how many engineering aids are employed by the several State highway departments. Much has been written about the current shortage of engineers, and of the States' needs and requirements in connection therewith, but certainly an accurate tabulation of present engineering employees is prerequisite to a solution of the problem. Under the circumstances, a new and more definitive overall study is desirable, so that data for all States can be reported on a uniform basis.[3]

Qualifications of Engineers

Table 2 shows the number and qualifications of highway department employees classified as engineers in each of the six States selected for further study. The classification concept used here is of course different from the "professional engineer" concept of the 1954 study, and is quite revealing. It is first noted that the difference between the 2,114 engineers of table 2 and the 1,589 total of table 1 is accounted for largely by two States—Mississippi and Washington. After some discussion with the appropriate State personnel, it was discovered that for the 1954 study Mississippi had reported only registered engineers, while Washington had reported only engineers of the associate or higher grades, omitting the junior and assistant grades. Each of the other States had reported all engineers classified as such, regardless of their professional qualifications or grades.

Thus, the reported total of 18,034 engineers employed by all State highway departments may be either high or low, depending on what

Table 2.—Number and qualifications of State highway department employees classified as engineers in six selected States

State	Civil engineer graduates and registered		Civil engineer graduates only		Registered engineers only		Neither graduates nor registered engineers		Employees classified as engineers		Civil engineer graduates		Registered engineers	
	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent
Mississippi	82	38	4	2	34	16	96	44	216	100	86	40	116	54
Nebraska	28	12	25	11	71	30	111	47	235	100	53	23	99	42
Oregon	63	13	77	16	39	8	302	63	481	100	140	29	102	21
Vermont	32	21	33	22	20	13	67	44	152	100	65	43	52	34
Washington	63	10	124	20	56	9	384	61	627	100	187	30	119	19
Wisconsin	115	29	98	24	55	13	135	34	403	100	213	53	170	42
Total	383	18	361	17	275	13	1,095	52	2,114	100	744	35	658	31

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[3] The Highway Research Board is undertaking a census of the State highway department engineering employees as of March 1, 1956. Results should be available by July 1.

Table 3.—Number of employees classified as engineers[1] per $1 million of capital outlay, according to qualifications, in six selected States

State	Civil engineer graduates and registered	Civil engineer graduates only	Registered engineers only	Neither graduates nor registered engineers	Employees classified as engineers	Civil engineer graduates	Registered engineers
Mississippi	3.0	0.2	1.3	3.6	8.1	3.2	4.3
Nebraska	1.5	1.4	3.8	6.2	12.9	2.9	5.3
Oregon	1.3	1.9	.8	7.1	11.1	3.2	2.1
Vermont	6.0	6.3	3.8	12.9	29.0	12.3	9.8
Washington	1.3	2.7	1.1	8.3	13.4	4.0	2.4
Wisconsin	3.3	2.9	1.2	3.8	11.2	6.2	4.5
Average	2.2	2.1	1.4	6.3	12.0	4.3	3.6

[1] Excluding those assigned to maintenance.

is wanted. If the ratio which exists between tables 1 and 2 is applied to all States, the figure becomes approximately 24,000 on a classification basis. If, on the other hand, the professional engineer concept is adhered to, the 18,034 figure is probably much too high; the actual number will depend on how the term "professional engineer" is defined.

This matter of definition is obviously an important one. Referring again to table 2, it is noted that 52 percent of the employees classified as engineers are neither civil engineering graduates nor registered professional engineers, the percentage varying from 34 to 63 among the six States. Also, only 35 percent of such employees are civil engineering graduages; only 31 percent are registered professional engineers; and only 18 percent are both civil engineering graduates and also registered professional engineers.

If an engineer is defined as a civil engineering graduate and/or a registered professional engineer, only 48 percent of those employees now classified as engineers could qualify according to table 2. Is this a reasonable definition, or does it do an injustice to the other 52 percent of the employees? Should these employees continue to be classified as engineers, or should they be reclassified and placed in high-grade engineering-aid classifications, which may not even exist at present? Granted that some of them may eventually attain registration, should our interest be directed toward potential or present qualifications?

In any event, table 2 presents some of the most important findings of the six-State study. It indicates that the engineers included by the States in the 1954 study were not reported on a uniform basis, and therefore raises a question as to the significance of any previously reported figure for the total number of engineers employed by the State highway departments. Also, by pointing out variations in the qualifications of employees classified as engineers, the tabulation demonstrates the need for a more exact definition of the term "engineer" and suggests a definition which might be usable; that is, a civil engineering degree and/or registration. Possibly this definition should be broadened to include the small number of mechanical, electrical, and other engineers engaged in State highway work.

Table 4.—Engineering personnel employed and number of engineering aids employed per engineer in six selected States

State	Engineering personnel employed		Engineering aids per engineer employed
	Engineering aids	Engineers	
Mississippi	459	216	2.1
Nebraska	237	235	1.0
Oregon	394	481	.8
Vermont	27	152	.2
Washington	376	627	.6
Wisconsin	606	403	1.5
Total	2,099	2,114	1.0

Now, what about the number of engineers employed per million dollars of capital outlay, on the basis of the information reported in the six-State study? Table 3 presents this information, according to the qualifications of the employees classified as engineers. This table was prepared by using the information presented in table 2, but excluding employees assigned to maintenance, with the capital outlays shown in table 1. The last column of table 1 is based on the 1954 study, while table 3 is based on the later six-State study.

According to table 3, the total number of employees classified as engineers per million dollars of capital outlay varies from 8.1 in Mississippi to 29.0 in Vermont. This is still a wide range, although not so extreme as that indicated by table 1. The principal differences are found in Mississippi and Washington, and are due to the substantially higher number of engineers reported for those States in table 2. Again referring to table 3, it should be noted that for the four States other than Mississippi and Vermont the total number of engineers per million dollars of capital outlay varies only from 11.1 to 13.4.

Similar variations exist for each of the several categories of engineers included in table 3. For those engineers who are both civil engineering graduates and registered, for example, the range is from 1.3 to 6.0; for all civil engineering graduates, the range is from 2.9 to 12.3. Although these ranges for the separate categories are less extreme on an absolute basis than is the range for total engineering employees, they are in most cases more extreme on a percentage basis. In all cases the upper extreme is represented by Vermont.

Since these rather wide variations still exist after the data reported by the several States have been put on a comparable basis, it is necessary to look further for an explanation. The next point of inquiry, then, is the number of engineering aids employed by the several State highway departments. Does the number of such aids vary directly or inversely with the number of engineers? If the latter, there is a possible explanation for the variations which exist with respect to the number of engineers employed per million dollars of capital outlay.

Relation of Engineering Aids to Engineers

There is no particular problem in defining the term "engineering aid", because the number of civil engineering graduates and/or registered engineers classified as engineering aids is insignificant. There is a problem of nomenclature, however, as to whether those employees who complement the engineers shall be called engineering aids, subprofessional employees, technicians, or something else. Since there is no obvious answer, they are called engineering aids here.

Table 4 shows the number of engineering aids employed and the number employed per engineer for each of the States included in the six-State study. It is noted, first, that the total number of engineering aids employed in all six States (2,099) is approximately the same as the number of engineers employed (2,114), so the ratio of engineering aids to engineers is 1.0. Incidentally, this ratio can be compared with a published overall ratio of one technician to 2.5 engineers for all fields of engineering and for all types of engineering endeavor.[4]

In the individual States, however, the ratio of engineering aids to engineers varies from 0.2 in Vermont to 2.1 in Mississippi. The significant fact here is that the low ratio exists in Vermont which has the highest number of engineers per million dollars of capital outlay, while the high ratio exists in Mississippi which has the lowest number of engineers per million dollars of capital outlay. Apparently there is some sort of an inverse relation between engineering aids and engineers.

This relation is demonstrated further by table 5, which shows the number of engineers and engineering aids employed per million dollars of capital outlay. Column 1 of this table is taken directly from table 3. Column 2 was obtained by combining data from tables 1 and 4; no correction was made for maintenance employees, because very few engineering aids are engaged in maintenance activities.

Table 5.—Engineers and engineering aids employed per $1 million of capital outlay in six selected States

State	Engineering employees per $1 million of capital outlay[1]		
	Engineers	Engineering aids	Total
Mississippi	8.1	17.5	25.6
Nebraska	12.9	14.4	27.3
Oregon	11.1	9.8	20.9
Vermont	29.0	5.2	34.2
Washington	13.4	8.1	21.5
Wisconsin	11.2	18.1	29.3
Average	12.0	12.5	24.5

[1] See table 1 on p. 30 for capital expenditures.

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[4] *Engineers and technicians* (editorial), Engineering News-Record, vol. 155, No. 21, Nov. 24, 1955, p. 164.

Table 6.—Relative rank of six selected States according to capital expenditures, number of engineering employees, and number per $1 million of capital outlay [1]

State	Capital outlay	Number of engineers		Number of engineering aids		Number of engineers and engineering aids	
		Total	Per $1 million of capital outlay	Total	Per $1 million of capital outlay	Total	Per $1 million of capital outlay
Mississippi	4	5	6	2	2	4	4
Nebraska	5	4	3	5	3	5	3
Oregon	2	2	5	3	4	3	6
Vermont	6	6	1	6	6	6	1
Washington	1	1	2	4	5	2	5
Wisconsin	3	3	4	1	1	1	2

[1] The highest amount or ratio is ranked 1 in each case, the next highest 2, etc.

It is evident from table 5 that combining engineers and engineering aids reduces considerably the extreme variations in the number of engineering employees per million dollars of capital outlay. For engineers, the high figure is 358 percent of the low figure; for engineering aids, the high figure is 348 percent of the low. When the two are combined, however, the high figure is only 164 percent of the low figure.

It has been mentioned previously that program or capital outlay amounts may vary widely from year to year in a particular State, and in some States the number of engineering aids employed increases greatly during the construction season. In Nebraska, for example, the 1953 capital outlay was only $9.9 million, as compared with the 1954 figure of $16.5 million reported in table 1. In Wisconsin, the number of engineering aids employed practically doubles during the summer months. Thus, the ratios established cannot be considered as final or conclusive in any one instance, but do indicate a definite inverse relation between engineering aids and engineers.

This inverse relation explains at least partially the wide variations among the States with respect to the number of engineering employees per million dollars of capital outlay. Since the States use engineers and engineering aids in different proportions, a combination of the two is the best indication of engineering effort for a particular State. As previously mentioned, when States are compared on this combination basis, some of the extreme variations are eliminated.

It remains, then, to relate the number of engineers and engineering aids employed to both program characteristics and to management practices and procedures. Perhaps this analysis will explain further the variations among the States as to number of engineering employees per million dollars of capital outlay. In any event, it would be unrealistic to expect that these variations could be explained away entirely.

Program Characteristics

In relating the number of engineering employees to program characteristics, perhaps the most obvious feature to be considered is that of program amount. Other characteristics which can be analyzed include the relative amounts of rural and urban work, the relative amounts of "surfacing only" and all other work, and the average length of projects in each of the several work categories.

Table 6 shows the relative rankings of the six States selected for study with respect to program amounts and number of engineering employees. The rankings indicated in the first column are based on the program amounts reported in table 1, while those in the remaining columns are based on data in tables 2–5. Although not apparent at first glance, certain relations between program amounts and number of engineering employees are evident after some study of table 6.

The most direct relation is that between program amounts and total number of engineers; Washington, Oregon, Wisconsin, and Vermont rank 1, 2, 3, and 6, respectively, in each instance, while Mississippi and Nebraska rank 4 and 5 in one case and 5 and 4 in the other. Similar, but less direct relations exist between program amounts and total number of engineering aids and between program amounts and total number of engineers and engineering aids combined. These relations are not surprising, of course, since it is only reasonable that the number of engineering employees should increase with the size of the program.

As to relations between program amounts and number of engineering employees per million dollars of capital outlay, none is clearly evident from table 6. If any relations do exist, they are inverse relations, and this too is as it should be. It is logical that the number of engineering employees per million dollars of work should decrease, at least to some extent, with an increase in the amount of work.

The relations between total number of engineering employees and number per million dollars of capital outlay, as indicated by table 6, are quite surprising. For engineers alone and for engineers and engineering aids combined, these relations are not very definite; if they exist at all, they appear to be inverse. In the case of engineering aids, however, a direct and quite positive relation exists. No obvious explanation for this difference suggests itself, but again the variety of factors which may affect the ratios for the number of engineering employees per million dollars of capital outlay is emphasized.

In any event, table 6 establishes a direct relation between program amounts and number of engineering employees. As previously indicated, this relation is not unexpected, and in fact almost has to exist, since no relation or an inverse relation would be entirely illogical. The other relations indicated by table 6, if they exist at all, are not very positive, and probably are not significant.

Table 7 presents information on the relative amounts of rural and urban work and of "surfacing only" and all other construction work performed by the States. The program amounts on which this table is based are different from those indicated in table 1 and ranked in table 6, and in some cases the period covered is other than the 1954 calendar year. The program selected was one which could be conveniently analyzed by the State concerned, and generally the analysis is based on contracts awarded.

The theory here is, of course, that States doing a high percentage of urban work or a low percentage of "surfacing only" work will require more engineers than other States. The assumption is that urban projects and projects involving work other than surfacing or resurfacing only require more engineering effort. Whether this theory can be demonstrated depends on a comparison of table 7 with tables 2–5.

The rural percentage of total program costs varies from 75 in Washington to 89 in Mississippi, according to table 7. It follows then that the urban percentage varies from 25 in Washington to 11 in Mississippi. Theoretically, Washington should be using more and Mississippi fewer engineers than any other State included in the study. Table 5 indicates that Mississippi in fact uses fewer engineers per million dollars of capital outlay than any other State included in the study, and that Washington uses more than any other State except Vermont.

A comparison of the other data presented in table 7 with the information presented in earlier tables reveals no additional relations. In fact, such a comparison raises some questions about the direct relation which appears to exist between the urban percentage of total program costs and the number of engineers employed per million dollars of capital outlay. Washington, for example, does the greatest amount of urban work on a percentage cost basis, but also does the greatest amount of surfacing or resurfacing work on the same basis. Generally, it would appear that program costs are a more valid indication

Table 7.—Construction in rural areas and construction other than surfacing or resurfacing related to the total program

State	Construction in rural areas related to total program		Construction other than surfacing or resurfacing related to total program	
	Construction cost	Miles constructed	Construction cost	Miles constructed
	Percent	*Percent*	*Percent*	*Percent*
Mississippi	89	98	79	68
Nebraska	82	99	76	49
Oregon	88	94	85	73
Vermont	87	94	100	100
Washington	75	97	75	54
Wisconsin	80	96	85	59
Average	82	97	82	60

Table 8.—Average length of construction projects in six selected States

State	Average length of construction projects			
	Surfacing only		All other construction	
	Rural	Urban	Rural	Urban
	Miles	*Miles*	*Miles*	*Miles*
Mississippi	7.1	1.1	5.3	1.2
Nebraska	6.8	1.3	6.8	1.0
Oregon	2.8	1.4	1.5	1.2
Vermont	----	----	2.9	2.8
Washington	11.2	8.3	5.1	1.0
Wisconsin	5.0	.6	4.3	1.1
Average	6.5	2.0	3.8	1.1

of engineering effort than is program mileage, regardless of type of work.

The final program characteristic to be analyzed is that of project length. Table 8 presents information as to the average length of construction projects for both "surfacing only" and all other types of construction, and for rural and urban projects. Although these data are interesting, no relations between project length and number of engineering employees are evident.

With respect to program characteristics in general, there appears to be a direct and quite definite relation between program amount and total number of engineering employees, whether engineers alone, engineering aids alone, or both are considered together. Also, there appears to be a direct relation between the urban portion of total program costs and the number of engineers employed per million dollars of capital outlay, but a corresponding relation does not seem to exist in connection with engineering aids or engineers and engineering aids combined. Other relations either do not exist or are not apparent, possibly because of the relatively few States included in the study, and also because of certain intangibles such as climate, terrain, soil characteristics, and other factors which cannot be evaluated.

Management Practices and Procedures

Since this is primarily a classification study, a full-scale analysis of management practices and procedures was not attempted. Nevertheless, some attention was given to management practices and procedures in each State studied, and their possible effects on engineering manpower requirements were discussed with State personnel. Incidentally, it might be mentioned that classification is itself a management practice.

Of the six State highway departments included in the study, all but one are directed by commissions. In Mississippi, Oregon, and Vermont these commissions are 3-member part-time bodies, while in Washington the commission is a 5-member part-time body. In Wisconsin the commission is also a 3-member body, but serves on a full-time basis. The Nebraska Bureau of Highways is directed by a single executive, the State Engineer. Probably the type of directing organization has little effect on engineering manpower requirements in these six States.

In Oregon and in Washington one individual serves both as chief administrative officer and also as chief engineering officer. In each of the other four States, the chief administrative officer and the chief engineering officer are separate individuals. The commission chairman serves as chief administrative officer in Wisconsin, and in Nebraska the State Engineer serves as chief administrative officer, but in none of the other States does a member of the directing organization serve as either chief administrative or chief engineering officer. Again it seems that these differences have little effect on engineering manpower.

With respect to structural organization below the directing level, there are of course differences in the individual States, but all have field districts or divisions responsible to some extent for both construction and maintenance activities. All are decentralized to some degree, and all have the usual complement of central office bureaus. In spite of this overall similarity in structure, there are differences in operating methods which are significant in connection with a study of engineering manpower requirements.

In Mississippi, for example, few engineers but many engineering aids per million dollars of capital outlay are employed, and operations are decentralized to a considerable extent. Location surveys are made by field district personnel, and the project engineer lays a tentative grade on the project plan, which is used by the central office employees, if possible. Relatively inexpensive designs can be used because of the relatively light traffic in the State, and shortcuts are used in the design process itself. Standard plans are used for over 90 percent of all bridges, and bridge construction and maintenance are done by the regular construction and maintenance forces. Difficulty is experienced in hiring the higher grades of engineering personnel, but the lower grades are more readily available.

In Nebraska, where the number of engineering employees per million dollars of work is just slightly above the average for the six States included in the study, location is a central office function. Also, design has been a centralized function, but recently some of this work has been done in the field, and eventually the field will probably design everything except structures. All construction work, including Federal-aid secondary projects, is done to the same standards and specifications, and probably more inspecting is done than in most States. Also, State employees do most of the engineering work on Federal-aid secondary jobs. With respect to personnel, detailers are more critical than designers.

Oregon and Washington are alike in many ways, but present a marked contrast with respect to operating procedures. In Oregon, operations are largely on a centralized basis, and the field organization is somewhat rigid. In Washington, operations are decentralized to a considerable degree, and the field organization is quite flexible. Nevertheless, these two States have relatively few engineering employees per million dollars of capital outlay, and their records in this respect are the best of any of the six States studied, as is indicated by table 5.

Vermont employs more engineers and fewer engineering aids per million dollars of capital outlay than any of the other States included in the study, and there is no obvious explanation for the particularly high number of engineers employed. Probably a combination of factors, including an extreme climate, a difficult terrain, and a small program amount, is responsible. State highway operations are more centralized than in most other States, but the field districts do a considerable amount of engineering on State-aid work which is not reflected in the program amount. Also, many employees classed as engineers do work which is done by engineering aids in other States, although this is because of the classification system, rather than in spite of it. Incidentally, the State reports that because of this new classification system the shortage of engineers is no longer a problem.

Wisconsin uses relatively few engineers, but more engineering aids than any other State studied, per million dollars of capital outlay. More time and attention are given to management practices and procedures than in most other States, and the organization plan is one of decentralization with centralized controls. Planning is emphasized, and engineers are being used more and more in supervisory capacities rather than for operations. A trained corps of engineering aids or technicians is being developed, and these nonengineers are being substituted for engineers wherever possible. Expanded programs are being handled without any appreciable increase in the number of engineering employees.

It appears that in any particular State there are some management practices which promote overall efficiency and the efficient use of engineering personnel, and other practices which perhaps tend to be wasteful of personnel. Also, it might be noted that a procedure which promotes overall efficiency may at the same time appear to be somewhat wasteful of personnel. In any event, it is extremely difficult to correlate the ratio of engineering employees to capital outlays with particular management practices and procedures, and perhaps this ratio is determined primarily by other factors.

Wisconsin, for example, has emphasized the use of engineering aids, and its ratio of engineering aids to capital outlay is high. Vermont has adopted a classification plan which makes it possible for employees with only a practical background to attain relatively high-grade engineering classifications without taking examinations, and has the highest ratio of engineers to capital outlay of any State studied. Washington has accomplished the increased work resulting from a bond program without a proportionate increase in engineering personnel, while Oregon increased its engineering forces by only 40 percent over a 5-year period to handle a bond program which doubled the construction program. Probably, then, a major factor in the efficient use of engineering personnel is the necessity to get along with what is available.

The Shortage of Highway Engineers and How to Overcome It

OFFICE OF THE ASSISTANT TO THE COMMISSIONER
BUREAU OF PUBLIC ROADS

Reported[1] by ROBLEY WINFREY, Chief,
Personnel and Training Office

THE past 2 years have brought forth many serious discussions about the engineering manpower necessary to carry on a greatly increased program of capital outlay for highways. Current shortage of engineers has also been accorded its due attention. Month-by-month the highway departments are finding additional ways to do more highway work without hiring additional engineers. Highway department managements are tackling manpower needs with the same energy, alertness, and success that have always characterized their attack on the technical problems encountered in the effort to keep up with the ever-increasing demand for highway usage.

But what are the engineering needs? How can these needs be met successfully? Let's first look at highway engineering employment. What is it? How much do we have?

Engineers and Engineering Aids Employed

An engineer is what definition says he is. Each person, each civil service system, and each hiring organization has defined the engineer somewhat to fit local concepts and local needs. This lack of an accepted definition has prevented determining the number of engineers employed in the highway industry, public and private. The hiring of employees to assist the engineer under such titles as surveyors, draftsmen, computers, inspectors, and laboratory analysts is the primary reason for lack of uniformity in definition. For the purpose of this article these engineer assistants are grouped into one class and called engineering aids, sometimes known as subprofessionals, preprofessionals, nonprofessionals, and technicians.

The word "engineer" is used herein to refer to an employee or potential employee who is a registered professional engineer under the law of his State; eligible for such registration, but because of not being required to do so, has not applied for registration; a graduate at the level of bachelor of science or higher of an accredited college course in engineering; and those employees who are classified as professional engineers by the State highway department or under a local merit or civil service system. In the latter group, there will be found in certain States, employees classified as engineers who in other States are classified as engineering aids.

No census of highway employment exists on the basis of these or other known definitions; therefore, only estimates of the number of engineers employed are available. According to Campbell and Schureman (*3*)[2] the State highway departments in September 1954 were employing 18,034 engineers and had need for 3,990 more. Danner, in his committee report (*4*) indicated the employment of engineers by State highway departments to be 14,204 as of November 30, 1955, for 30 States. Prorating by the Campbell-Schureman data, Danner's total becomes 20,440 for the 48 States.

Lewis (*9*) reported further on the classification and employment of engineers in six States. The six-State study shows that the employment reported by Campbell and Schureman was controlled by different interpretations which were placed on the questionnaire sent out by the Highway Research Board as well as by differences in State classification. Another factor that has not been controlled is date of reporting. Employment varies considerably with seasons and with available funds, so these two studies are not directly comparable. Further, engineering aids or their equivalent were not reported in the Campbell-Schureman study and the extent they are included is unknown.[3]

In summary, the conclusion must be that the number of engineers and the number of engineering aids employed by the 48 State highway departments are not known. For purposes of later discussions, a total of 20,000 engineers and 20,000 engineering aids are used. Of the 20,000 engineers, 1,300 are assumed to be primarily maintenance engineers, leaving 18,700 engaged in duties pertaining to construction.

Lewis (*9*) reported for six States (Mississippi, Nebraska, Oregon, Vermont, Washington, and Wisconsin) that, exclusive of maintenance engineers, these States were employing 12 engineers per million dollars of capital outlay and that the ratio of engineers to engineering aids was 1 to 1. His report further shows that if the number of engineers per million dollars of construction is relatively high the number of engineering aids per million dollars of construction is relatively low. The reverse is also true.

Campbell and Shureman (*3*) found that the ratio of engineers per million dollars of construction varied from 2.0 to 28.2 State-to-State with a median ratio of 7.2. Since it is known that the data were not consistently reported by the States, the wide range of 2.0 to 28.2 and the median value of 7.2 are not of any determinable reliability.

The six States studied by Lewis were not chosen as being representative of the 48 States, but for other reasons associated with his objectives. But for purposes of illustration only, the assumption of 7.2 engineers per million dollars of construction outlay is used.

The 1955 construction expenditures by the State highway departments totaled about $2.5 billion, which at 7.2 engineers per million dollars indicates a requirement of 18,000 engineers.

For a greatly enlarged highway construction program the ratio of engineering employment to construction dollars could be reduced, and because of scarcity, will be reduced. But assuming the ratios of 7.2 engineers and 7.2 engineering aids for each million dollars of construction money available, an increase in the program of $2 billion annually would require an additional employment of 14,400 engineers and 14,400 engineering aids. Whether the increased program were handled wholly by the States, or by the States and consultants, the manpower requirements would be approximately the same. This requirement is 80 percent of the current employment in the engineering operations. The 14,400 additional engineers are just not available, so how can the program be supplied with adequate engineering services? The following analysis presents an interesting five-way approach.

Meeting the Manpower Requirement

The total engineering services can be supplied for a highway construction program at an increased rate of $2 billion a year. Although the facts are not available from which a detailed breakdown can be presented, the following plan is both possible and reasonable.

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[1] This article was presented at the Ninth Northwest Conference on Road Building, University of Washington, Seattle, Feb. 1956.

[2] Italic numbers in parentheses refer to the list of references on page 36.

[3] The Highway Research Board is undertaking a census of the State highway department engineering employees as of March 1, 1956. Results should be available by July 1.

First, adjust present employment procedures, operations, management policies, and assignments to those selected from the 48 States that have been successful in operating with the least amount of engineering services.

Second, shift as many duties as possible from engineers to engineering aids, and from engineers and engineering aids to fiscal, clerical, and administrative employees.

Third, adjust personnel management policies and employee relations to effect measures which will reduce the number of resignations and which will increase general productivity.

Fourth, operate in-service training and educational programs as a means of increasing quantity and quality of production of employees and of upgrading their classification.

Fifth, plan and execute an organized program of recruiting manpower and of selling highway employment to college and high school graduates who now seek other employment. Also, organize to hire from the large number of college students who drop out of college short of graduation.

There is not opportunity today to go into detail of what can be accomplished under step 1. Reports by Campbell (*2*) and Schureman (*10*) mention the following possibilities which have been achieved by one or more States:

1. Use aerial photographs and photogrammetric methods in location studies and mapping (50 to 80 percent saving in highway department manpower).
2. Identify soil types from aerial surveys.
3. Compute coordinates, error of closure, areas, and other survey calculations by automatic computing machines.
4. Adopt new surveying instruments of improved design. A new self-leveling level will cut time of leveling by 50 percent.
5. Adopt a State coordinate system of survey control.
6. Use standard designs for elements of the overall plans for highways, small structures, and bridges.
7. Use high-speed electronic computers in stress analysis.
8. Eliminate duplicated and unnecessary detail on construction plans, and use supplementary sheets for standardized items.
9. Use typing and acetate sheets instead of hand lettering on plans.
10. Use pencil drawings rather than "inked tracings" and use photographic methods of reproduction instead of blueprinting.
11. Compute earthwork quantities by business machines rather than by hand plotting.
12. Compute earthwork quantities from aerial photographs.
13. Centralize computations, estimating, pricing, and analysis of bids.
14. Increase length of roadway projects per contract.
15. Save engineers by increasing the number of survey parties supervised by one engineer and increase the number of construction projects handled by a resident engineer.
16. Streamline the whole process of highway design and construction to eliminate duplicated effort, unessential checking and reviews, and assign as many operations as possible to workers skilled in special operations.
17. Use mobile communication systems for field control.
18. Use many of the 1,500 separate items collected by Campbell (*2*).
19. Adopt many other items yet in experimental stages and concentrate on the development of new ideas.

By achieving the maximum improvement in the overall total engineering processes, the number of engineers required per million dollars of construction could be greatly reduced for the proposed total program.

In the second step, many of the duties now performed by engineers can be assigned to engineering aids. In this process, suitable training will need to be given the engineering aids as mentioned in step 4. The potential saving of the manpower of engineers by this process should result in the employment ratio of 1 engineer to 2 engineering aids. In another 10 years a ratio of 1 to 3 might be achieved. Certainly, the State highway department engineers could quickly devise work processes, and so arrange the assignment of duties that each engineer could direct the work of two aids.

The third and fourth steps, personnel management and in-service training, should be combined to effect an overall increase in productivity and to permit a reduction of at least 10 percent in engineering services per million dollars of construction.

Engineering Services Required for an Accelerated Program

If all the new devices, methods, and procedures known to be available were used and the best personnel management practices were followed, engineering services for construction could be reduced from 7.2 engineers and 7.2 engineering aids to 5 engineers and 10 engineering aids per million dollars of construction.

Assuming the use of the ratio of 5 engineers and 10 engineering aids for the new construction program of $2 billion in addition to the current State highway department program of about $2.5 billion, the total requirement for $4.5 billion would be 22,500 engineers and 45,000 engineering aids. These two estimates compare with the previous estimate of 18,700 engineers and 20,000 engineering aids presently employed by State highway departments. The necessary increases would be 3,800 engineers and 25,000 engineering aids. Recruiting these numbers should be possible under a planned program and with determination to do so.

This analysis results in an estimate of additional engineers far less than the number many highway engineers have in mind. The actual figures must await a reliable census of current employment. However, strong support must be accorded to the basic principle proposed—the number of engineers required per million dollars of construction can be reduced materially from the existing ratio (whatever it may be) by applying the new know-how. And if the new enlarged program is to be achieved, the ratio *must* be cut about 50 percent.

Recruiting for Additional Manpower

The additional personnel required for the increased highway construction program should be obtainable from the following sources and by the methods indicated. An important factor is that at least 3 years would be available before the full number of engineers would be needed.

1. From increased numbers of graduating engineers, brought on through an upward trend in enrollment and by a more active recruiting program, some 3,000 engineers could be hired in the next 3 years. In 1955 the engineering colleges graduated 3,868 civil engineers with bachelor degrees. In 1958 the number should increase to about 5,500. In 1955 the State highway departments hired about 550 graduates. By organized and determined recruiting, certainly an average of 1,000 a year is possible from 1956 to 1958.
2. By recruiting directly for the high school graduate under a planned program, a total yearly employment of 15,000 could be achieved. In 1954–55 the Nation's high schools graduated 1,399,300 boys and girls.
3. Close attention should also be directed to the hiring of engineering students who do not complete their college program. Many colleges graduate only 30 to 50 percent of those students who enroll as freshmen.
4. Employment can be greatly increased by bringing into highway employment, engineers, technicians, aids, and other types of trained personnel who can easily adapt their training and experience to highways. Branches other than civil engineering should be tapped for available employees.
5. When the total employment of civil engineers by all levels of government and private firms is considered, there is a large employment to draw upon. Within these other government employments it will be possible to effect many of the same savings of manpower as suggested for the State highway departments. Adoption of such improved methods should make it possible to release several hundred engineers for highway work without unduly handicapping normal operations of the other agencies and firms.
6. By utilizing services of private engineering firms for the most difficult and unusual type of facilities, by contracting out all possible nonengineering services but including aerial surveying, map making, printing of plans, and special rights-of-way functions, the equivalent of thousands of employees can be added to the States' services.
7. Through improved personnel management, the rate of resignation can be reduced materially. The rate of resignation for civil engineering graduates as experienced by 26 States was reported (*12*) to be 48.8 percent in the first 14 months for the 1952 class and 19.0 percent in 2 months for the 1953 class. Further, employees by better placement can materially increase their production.
8. Much can be gained by improving employee utilization through research and experimentation into human relations and human productivity. Where does it get the highway industry to spend millions of dollars each year on research on use of materials,

Table 1.—Application of the ratio of one engineer to two engineering aids to the employment practices in three States following an 80-percent increase in funds over the 1955 construction program

	Employment in 1955 with a $50-million construction program			Employment with a $90-million construction program [1]			Change in employment with increase in program		
	State A	State B	State C	State A	State B	State C	State A	State B	State C
Number of engineers employed	700	500	350	450	450	450	−250	−50	+100
Number of engineering aids employed	400	500	700	900	900	900	+500	+400	+200
Engineering aids per engineer	0.57	1.00	2.00	2.00	2.00	2.00	-----	-----	-----
Engineers per $1 million of construction	14	10	7	5	5	5	−9	−5	−2
Engineering aids per $1 million of construction	8	10	14	10	10	10	+2	0	−4
Total engineering employees per $1 million of construction	22	20	21	15	15	15	−7	−5	−6

[1] Based on a $40-million increase in the 1955 construction program for three typical States, and an employment rate of 5 engineers and 10 engineering aids per million dollars of construction.

machines, and money and not a dollar on how to understand those human factors which in the end control and utilize the product of such physical research? Research on how to use the power of man can greatly alter our concept and utilization of manpower. With such new concept we should not hesitate to embark on a program of construction double the current rate of expenditure.

9. Operation of essential training programs for both college civil engineering graduates and engineering aids should be considered. See references (*8*), (*12*), (*13*), and others, for suggested programs and States now operating successful programs. Notable success in the training of high school graduates is being achieved by Connecticut, Iowa, Michigan, Nebraska, and Wisconsin (*13*). Many States have cooperative arrangements with colleges for the alternate college-work plan by which one group of college students is exchanged with a second group at the highway department on a scheduled basis.

10. Educational programs and in-service training programs can be used as a means of preparing present employees for upgrading and qualifying them for professional positions. This plan should make available a considerable number of engineers and the vacancies created by their promotions can be filled by younger and less experienced new employees.

There is no basis for making an estimate of the overall increase in employment of engineers and engineering aids that would result by full application of these 10 methods. Over a 3-year period, however, employment certainly would approach the estimated total need of 22,500 engineers and 45,000 engineering aids.

Ratio of One Engineer to Two Engineering Aids

An application of the plan to typical State conditions is shown in table 1. For a total national increase in the construction program of $2 billion each State (A, B, and C) might receive an increase of $40 million annually. The total program for each State would then be $90 million a year. At the rate of 5 engineers and 10 engineering aids per million dollars, each State (A, B, and C) would then require 450 engineers and 900 engineering aids. The resulting changes in employment are shown in the last three columns of the table.

It is not likely that States A and B would decrease the number of engineers employed when practically doubling their construction program. But the indicated increase in the number of engineering aids could be expected. In view of the discussion at the beginning of this article about the census and classification of engineering employment, it can be said that States A and B may have a large number of aids classified as engineers in comparison with State C. But the three States have 22, 20, and 21 engineering employees per million dollars of construction funds. The goal of 15 total—5 engineers and 10 aids—calls for a reduction of 7, 5, and 6, respectively.

Certainly such reduction in the ratio of engineering employment is possible. By adopting all that is currently known about how to conserve engineering manpower and all that can be discovered in the next 3 years, this proposed reduction in ratio (increase in productivity per employee) is reasonable, practical, and possible. In fact, should the Congress authorize the funds for an accelerated program such adjustments in the utilization of engineering services will come about of necessity.

Highway engineers have always successfully met their technical challenges. We can now have confidence that they will again meet their technical challenge and their human challenge as well.

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF APRIL 30, 1956

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$3,608	$18,367	$9,932	304.0	$8,876	$5,015	61.6	$43,015	$22,559	642.5	$70,258	$37,506	1,008.1
Arizona	1,530	6,282	4,722	104.0	3,741	2,867	46.2	10,557	7,867	106.8	20,580	15,456	257.0
Arkansas	7,950	16,787	8,414	527.8	6,372	3,343	56.6	22,561	11,589	471.9	45,720	23,346	1,056.3
California	3,787	23,345	12,645	219.7	15,481	8,187	17.9	155,393	79,666	295.4	194,219	100,498	533.0
Colorado	15,893	6,265	3,528	84.6	1,993	1,156	23.0	17,562	9,440	190.5	25,820	14,124	298.1
Connecticut	20,334	3,685	1,905	7.6	1,778	947	4.2	12,338	6,249	17.3	17,801	9,101	29.1
Delaware	3,999	3,825	1,922	40.5	1,534	792	21.1	6,170	3,089	50.7	11,529	5,803	112.3
Florida	6,498	11,753	6,356	141.1	11,758	6,191	47.6	36,603	18,753	369.5	60,114	31,300	558.2
Georgia	20,962	20,683	10,748	319.9	8,925	4,319	105.9	45,828	21,660	743.6	75,436	36.727	1,169.4
Idaho	6,071	5,455	3,476	87.9	4,154	2,685	68.1	12,352	8,078	149.1	21,961	14,239	305.1
Illinois	12,041	38,797	20,596	692.9	21,090	11,455	84.6	95,943	52,201	479.6	155,830	84,252	1,257.1
Indiana	23,127	23,775	12,572	177.5	13,368	6,896	147.4	45,096	24,800	144.1	82,239	44,268	469.0
Iowa	4,569	22,265	12,733	1,006.8	8,936	4,924	117.5	27,670	14,865	778.8	58,871	32,522	1,903.1
Kansas	6,838	15,069	7,827	906.1	9,645	5,115	179.5	22,238	11,402	816.1	46,952	24,344	1,901.7
Kentucky	12,697	8,270	4,169	183.9	1,906	1,140	4.8	40,812	21,649	587.0	50,988	26,958	775.7
Louisiana	8,606	14,589	7,302	86.2	3,302	1,658	1.6	48,236	23,159	482.8	66,127	32,119	570.6
Maine	3,089	11,265	5,855	93.6	86	86		15,895	8,186	107.9	27,246	14,127	201.5
Maryland	4,968	22,595	11,722	143.4	12,684	5,632	16.4	20,774	10,835	71.2	56,053	28,189	231.0
Massachusetts	13,521	12,353	6,167	18.5	13,556	7,342	8.9	41,460	20,041	62.2	67,369	33,550	89.6
Michigan	8,196	36,026	18,194	670.8	20,973	11,253	115.4	65,345	33,704	438.5	122,344	63,151	1,224.7
Minnesota	5,748	18,531	9,777	1,000.5	10,235	5,811	203.1	35,128	18,504	946.0	63,894	34,092	2,149.6
Mississippi	10,236	11,083	5,430	472.6	6,241	3,498	171.7	27,212	13,937	713.1	44,536	22,865	1,357.4
Missouri	3,548	34,826	17,453	1,457.2	1,121	557	1.3	79,657	41,732	1,196.5	115,604	59,742	2,655.0
Montana	12,693	10,999	6,644	201.9	7,358	4,500	137.5	24,903	15,540	432.8	43,260	26,684	772.2
Nebraska	14,170	11,027	5,878	318.8	4,869	2,590	101.6	32,889	17,318	965.2	48,785	25,786	1,385.6
Nevada	10,485	1,225	1,023	24.1	2,690	2,282	27.3	8,535	7,199	174.0	12,450	10,504	225.4
New Hampshire	5,746	1,938	1,175	10.9	1,724	860	7.9	8,387	4,321	53.1	12,049	6,356	71.9
New Jersey	21,293	10,364	4,785	54.6	6,361	3,297	11.8	33,270	15,433	43.6	49,995	23,515	110.0
New Mexico	5,852	3,335	2,133	40.5	4,121	2,621	80.9	12,260	8,082	208.3	19,716	12,836	329.7
New York	40,997	35,680	19,307	95.9	54,805	27,861	121.9	200,346	93,455	253.2	290,831	140,623	471.0
North Carolina	8,790	23,649	11,612	508.1	2,425	1,153	32.4	55,463	27,351	667.9	81,537	40,116	1,208.4
North Dakota	4,057	13,132	6,755	1,365.9	10,230	5,223	645.7	9,852	5,059	541.7	33,214	17,037	2,553.3
Ohio	16,658	54,541	28,425	226.6	11,497	6,280	30.1	81,889	40,391	123.0	147,927	75,096	379.7
Oklahoma	12,664	16,595	8,379	340.8	10,526	5,375	130.4	38,312	20,097	359.2	65,433	33,851	830.4
Oregon	3,607	2,400	1,410	35.3	2,870	1,758	27.9	26,228	16,129	255.1	31,498	19,297	318.3
Pennsylvania	31,380	10,913	5,807	31.6	44,399	20,863	38.5	103,446	53,129	368.0	158,758	79,799	438.1
Rhode Island	1,401	1,910	955	5.7	3,828	1,919	8.4	19,557	10,092	28.9	25,295	12,966	43.0
South Carolina	8,981	18,784	10,142	376.4	897	502	9.1	20,404	10,644	403.4	40,085	21,288	788.9
South Dakota	1,724	20,217	11,663	749.0	7,689	4,499	263.0	13,578	7,743	570.9	41,484	23,905	1,582.9
Tennessee	16,534	17,061	8,535	335.8	4,394	2,200	27.5	47,137	21,730	558.0	68,592	32,465	921.3
Texas	11,952	19,438	10,281	439.0	27,297	14,372	254.3	120,546	63,565	1,853.3	167,281	88,218	2,546.6
Utah	696	9,193	6,874	200.3	1,287	982	11.0	13,307	9,969	130.3	23,787	17,825	341.6
Vermont	3,801	3,041	1,520	32.5	113	59		10,252	5,298	93.5	13,406	6,877	126.0
Virginia	11,516	21;089	11,078	350.7	4,334	2,220	31.3	30,716	15,252	313.9	56,139	28,550	695.9
Washington	4,069	20,714	11,600	238.6	8,286	4,804	151.8	22,286	11,853	179.1	51,286	28,257	569.5
West Virginia	15,621	9,707	5,057	33.9	3,607	1,915	10.2	17,478	8,797	61.2	30,792	15,769	105.3
Wisconsin	8,767	19,744	9,971	275.6	9,203	5,014	79.7	51,721	25,391	503.5	80,668	40,376	858.8
Wyoming	658	7,431	4,918	107.8	1,257	809	25.3	16,997	11,133	337.8	25,685	16,860	470.9
Hawaii	3,792	3,651	1,805	10.1	3,723	1,834	3.5	4,251	1,888	5.8	11,625	5,527	19.4
District of Columbia	9,751	2,177	1,076	1.4	1,797	883	1.5	9,909	4,455	2.6	13,883	6,414	5.5
Puerto Rico	6,782	11,208	5,446	34.6				18,034	8,292	48.0	29,242	13,738	82.6
TOTAL	502,253	767,054	407,699	15,193.5	429,342	227,544	3,774.9	1,979,798	1,023,571	19,396.4	3,176,194	1,658,814	38,364.8

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A JOURNAL OF HIGHWAY RESEARCH

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PUBLIC ROADS,
U. S. DEPARTMENT
OF COMMERCE,
WASHINGTON

In this issue: A study of road-user and property taxes on selected vehicles. (Major truck routes U. S. 1 and 9 in the Newark, N. J., area. Pulaski Skyway in the foreground.)

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C. M. Billingsley, *Editor*

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Road-User and Property Taxes on Selected Motor Vehicles, 1956

BY THE RESEARCH REPORTS BRANCH
BUREAU OF PUBLIC ROADS

Reported by EDWIN M. COPE, Chief
Highway Statistics Section,
and LAURENCE L. LISTON,
Transportation Economist

The inseparable link between highway improvement and economic growth has focused public attention upon the financing of highways needed to accompany this growth. The great bulk of revenue for highway improvement is derived from road-user taxes. The purpose of this article is to provide data that make it possible to compare the total direct State and personal-property taxes on the road-user, as well as the yields from the individual levies. The data permit direct measurement of existing taxes so that comparisons can be made between vehicles, between States, and between different services.

This is done by showing the amounts of State road-user taxes and all direct personal-property taxes that would be paid in each State during one full year at rates in effect January 1, 1956, for each of 11 typical vehicles that represent significant points in the tax range.

For vehicles operating under uniform assumed conditions, the imposts vary in total amounts paid annually from $30 for a pickup farm truck to $4,379 for a five-axle diesel combination. The national average for a lightweight passenger car is $62. The familiar three-axle combination, in private use, pays on the average $334 in registration fees, $461 in motor-fuel tax, and $157 in property tax.

THE continued growth of the total economy could be retarded by lack of adequate highways. The key to adequate highways is, of course, money and since by far the largest part of revenues for highways is derived from road-user taxes, the road-user taxes of almost all jurisdictions are under study. The purpose of this article is to supply basic data on road-user taxes under existing legislation as of January 1, 1956, and to show to the extent practical the relation of road-user taxes to general property taxes; and in so doing, to give public authorities both direct measurement of the impact of the taxes on different vehicles, and to help in placing these taxes in perspective. This study is entirely objective and is not intended to indicate support for or opposition to any tax policy.

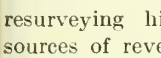
Twice before, the Bureau of Public Roads has presented studies that had the limited objective of making it possible to compare the road-user taxes of each State with those of other States on similar vehicles and similar services.[1] The previous studies were extensively used by public authorities, legislatures, and highway-user groups as basic material in their efforts to find revenues to close the gap between the rapidly increasing need for highways and existing highway facilities.

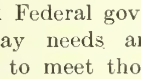
In the past 3 years, the number of vehicles has increased from 53 million to 63 million; and the States and Federal governments are resurveying highway needs and exploring sources of revenue to meet those needs. A great deal of study has been given to highway finance theories—ability to pay, the cost and value of the service, the incremental theory—and to the portion of the cost of highways that should be attributed to defense. But for effective work in tax theory and application, there is need for measurement of the amounts already being collected under existing tax rates. In the search for revenue, full consideration must be given to the present burden of all direct taxes, including property taxes on the vehicle owner.

The principal current State road-user revenues are derived from the gasoline tax and from registration fees on motor vehicles, but in some States a relatively low gasoline tax is accompanied by above-average registration fees, whereas in others, higher gasoline taxes are accompanied by low registration fees. In Georgia an automobile can be registered for $3 and the gasoline-tax rate is 6½ cents; in New York State it costs five times as much to register the same automobile, but the gasoline tax is only 4 cents. The result is that road-user taxes are about the same on an automobile in the two States, assuming average travel.

As in previous efforts, it is the purpose of this study to provide data that make it possible to compare the total direct State and personal-property taxes on the road-user, as well as the yields from the individual levies. Data presented here make possible direct measurement of existing taxes so that comparisons can be made between vehicles, between States, and between different services. This is done by showing the amounts of State road-user taxes and all direct personal-property taxes that would be paid in each State during one full year at rates in effect January 1, 1956, for each of a group of vehicles that represent significant points in the tax range.

This study is similar in many respects to the one published in 1953.[2] Extensive borrowing from the previous study has facilitated the preparation of this article. The similarity in the method of presentation should be an advantage to those who have used the previous material and will use the material presented here.


NOS. 1 & 2
PASSENGER CAR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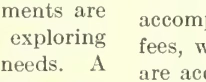
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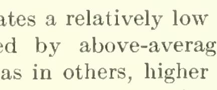
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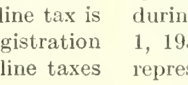
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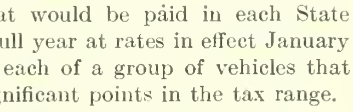
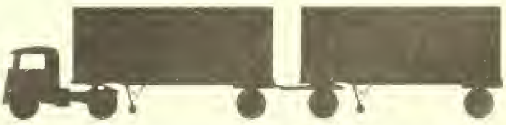

NO 7 - 3-AXLE TRACTOR-SEMITRAILER

NOS. 8 & 9 - 4-AXLE TRACTOR-SEMITRAILER

NO. 10 - 5-AXLE TRACTOR-SEMITRAILER


NO. 11 - 5-AXLE TRACTOR-SEMITRAILER AND FULL TRAILER

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[1] *Road-user and property taxes on selected motor vehicles, 1953,* by Edwin M. Cope and Richard W. Meadows. PUBLIC ROADS, vol. 27, No. 7, April 1953; also *State road-user and personal-property taxes on selected motor vehicles, 1950,* by R. W. Meadows and S. F. Bielak. PUBLIC ROADS, vol. 26, No. 2, June 1950.

[2] See footnote 1.

It is emphasized that the principal purpose of this article is to present the tax rates on a selected group of vehicles, and to make possible the comparison of the rates of one State with those of another.

The methods used and assumptions made in deriving the values presented should be carefully studied in order to avoid misuse through misunderstanding. That any vehicle would pay the precise annual amounts shown in the tabulations and figures is improbable, and this is particularly true for the large commercial vehicles. Consideration was given to reporting tax rates on the basis of ton-miles or vehicle-miles. The decision to show annual tax rates for selected vehicles has the following advantages: Such rates are more readily understood by the interested layman; many of the taxes and fees are levied on an annual basis; and using the same assumptions, the relative positions of the States are the same on either an annual or a vehicle-mile basis.

Previous studies evoked considerable discussion, and the authors received criticisms and suggestions. Some readers considered the annual mileages assumed for various vehicles to be too low, and others considered the mileages to be too high. That these criticisms had some effect is evidenced by the fact that the annual mileages assumed for the largest vehicles in this study are lower than those in the 1953 study.

Although there should be no difference between the relative positions of the States when comparing their vehicle taxes on an annual basis or comparing them on a vehicle-mile basis, the annual mileage assumed for the purpose of making the comparison can be important. If the annual mileage assumed is too low, those States with relatively large license or other annual fees, but no mileage taxes, are placed in an unfavorable light in comparison with States that have lower annual fees but do impose mileage taxes. On the other hand, if the assumed mileages are too great the States with mileage taxes are placed in an unfavorable light when compared with those that depend principally upon high annual fees for the taxation of heavy vehicles. It should not be assumed that the rates of taxation presented in this study are the actual amounts paid on any vehicle to a single State.

Nevertheless, the relative comparisons remain valid. The annual amount shown for each vehicle and each State is the amount that would be paid if all States had the same rate of tax as the State for which the data are presented, and if there were either complete reciprocity or proration of the registration fees, permanent fees, and similar imposts. Presentation of the tax data on a vehicle-mile basis would undoubtedly have lessened the possibility of the payments being misinterpreted, but it was felt that such procedure would greatly decrease the usefulness to legislative bodies, highway administrators, and the general public.

The Typical Vehicles

The bases for registering motor vehicles vary considerably among the States. As shown in table 1, the most common basis for registering passenger cars is the flat fee with no regard for other factors. Mississippi, in contrast, determines the levy by compounding a fixed fee and factors based on horsepower, gross weight, and vehicle age. The different bases for passenger-car registration are shown in figure 1, and those for trucks are given in figure 2. State gasoline-tax rates are represented in figure 3, and the States in which motor vehicles are taxed as personal property are shown in figure 4.

Eleven vehicles that are reasonably representative of the types and sizes that comprise the vehicle population were selected for the study: two passenger cars, four single-unit trucks, and five combinations. Their relative sizes and axle arrangements are shown in silhouette on page 37. Brief descriptions of the vehicles follow:

PASSENGER CARS:

No. 1.—A lightweight club coupe.

No. 2.—A mediumweight sedan.

SINGLE-UNIT TRUCKS:

No. 3.—A pickup truck registered for 4,800 pounds gross weight (commonly called a "½-ton" truck).

No. 4.—A stake truck registered for 12,500 pounds gross weight (commonly called a "1½-ton" truck).

No. 5.—A van registered for 19,000 pounds gross weight (commonly called a "2¾-ton" truck).

No. 6.—A tandem-axle dump truck registered for 40,000 pounds gross weight, of the type used in hauling coal, building materials, etc.

COMBINATIONS:

No. 7.—A three-axle tractor-semitrailer registered for 40,000 pounds gross weight, chosen so that it falls within the maximum length and weight limits of all States.

No. 8.—A four-axle tractor-semitrailer (tandem axles on the semitrailer), gasoline-powered, and registered for 50,000 pounds gross weight (No. 9 is the same vehicle, but with diesel power).

No. 9.—A four-axle tractor-semitrailer (tandem axles on the semitrailer), diesel-powered, and registered for 50,000 pounds gross weight (No. 8 is the same vehicle, but with gasoline power).

No. 10.—A five-axle tractor-semitrailer (tandem axles on both units), diesel-powered, and registered for 62,000 pounds gross weight.

No. 11.—A combination composed of a two-axle, cab-over-engine, diesel-powered tractor-truck, a single axle semitrailer and a two-axle full trailer; registered for 72,000 pounds gross weight.

The 40,000-pound combination (No. 7) would be permitted to register and operate in any State, and the 40,000-pound tandem axle, single-unit truck (No. 6) would be permitted to operate in any State except Washington, which has a flat limitation of 36,000 pounds on a truck with three axles. The Mississippi law that previously set a maximum of 37,650 pounds for such vehicles was revised in 1954 to permit 41,000 pounds. Because of axle spacing and axle-load limits, the 40,000-pound single-unit truck is on the border line of legality in some States with respect to registration and operation, but it has been included in order to show that point in the tax schedules. Although the 40,000-pound single-unit truck is found chiefly in specialized operations, the 40,000-pound, three-axle combination is in almost universal use and is undoubtedly the most common heavy unit.

When the 1953 study was made, three States (Kentucky, Pennyslvania, and Tennessee) prohibited vehicles as heavy as the 50,000-pound combination (vehicle Nos. 8 and 9), but since that time the Tennessee limit has been increased from 42,000 to 55,900 pounds, and the Pennsylvania limit has been increased from 45,000 to 60,000 pounds. Thus, Kentucky alone as late as January 1956 prohibited a 50,000-pound combination.

In order to compare taxes on vehicles similar in most respects except for type of fuel used, two 50,000-pound, four-axle combinations were included in the study, one with a gasoline engine and the other with a diesel engine. The assumption made that the gasoline combination operates 4 miles per gallon and that the fuel-consumption rate of the diesel combination is 6 miles per gallon is arbitrary, but is believed to be reasonable. It is recognized that the difference in the miles-per-gallon rates of two individual vehicles may fall substantially short of or exceed the assumed differential (a 50-percent difference in consumption may also be expressed as a one-third saving in fuel). Seven States now impose a higher tax on diesel fuel than on gasoline, and numerous other States are studying such a step with a view to reducing or eliminating the apparent tax advantage of the diesel-powered vehicle.

The 62,000-pound combination (No. 10) is a five-axle, diesel-powered, tractor-semitrailer combination 49 feet long, that can be operated in the West and a few Eastern States that are not contiguous, as shown in figure 5. Diesel power was specified for all of the heavier combinations, since it has had a tendency to replace gasoline power in the larger units where the heavier diesel engine may be efficiently employed. The 72,000-pound tractor semitrailer and full-trailer combination (No. 11) is permitted in Ohio and nine of the Western States.

In 19 States the registration schedule imposes a higher fee on a three-axle single-unit 40,000-pound truck than it does on a three-axle combination. (This is not true, however, in any State that registers a combination as a single unit.) An outstanding example of this is found in Montana, where the registration fee for the three-axle 40,000-pound gross vehicle weight straight truck is $400, but the total registration fee for the three-axle 40,000-pound gross vehicle weight combination is $100.

The inclusion or exclusion of data for a given vehicle in each State was determined solely on the basis of whether the vehicle, as

Table 1.—Basis for registration of passenger cars

Flat Fee:
- Alabama
- Arizona
- California
- Connecticut
- Kentucky
- Louisiana
- Nevada
- Ohio
- Oregon
- Pennsylvania
- Utah
- Virginia
- Washington
- Wisconsin
- Wyoming

Flat Fee and Age:
- Idaho
- Vermont

Horsepower Groups:
- Illinois
- Maine
- Massachusetts
- Missouri

EMPTY WEIGHT OR SHIPPING WEIGHT

Weight Groups:
- Delaware
- Florida
- Maryland
- Montana
- Nebraska
- New Jersey
- North Carolina
- Tennessee
- Texas
- District of Columbia

Weight Groups and Horsepower:
- Indiana

Weight Groups and Age:
- Minnesota
- North Dakota
- South Dakota

100-Pound Intervals:
- Colorado
- Michigan
- New York
- West Virginia

100-Pound Intervals and Age:
- New Mexico

100-Pound Intervals, Value and Age:
- Iowa

500-Pound Intervals:
- Georgia
- South Carolina

GROSS WEIGHT

Weight Groups:
- New Hampshire
- Rhode Island

Weight Groups and Horsepower:
- Arkansas

100-Pound Intervals:
- Kansas

Factory Delivered Price and Age:
- Oklahoma

Flat Fee, Horsepower, Gross Weight, and Age:
- Mississippi

Table 2.—Data used in computing State road-user taxes and property taxes on selected vehicles, 1956 registration year

Factors affecting taxation	Passenger cars		Single-unit trucks				Tractor-semitrailer combinations								Tractor-semitrailer and full trailer		
	No. 1: Light	No. 2: Medium	No. 3: Pickup	No. 4: Stake	No. 5: Van	No. 6: Dump	No. 7		No. 8 (gasoline)		No. 9 (diesel)		No. 10 (diesel)		No. 11 (diesel)		
							Tractor-truck	Semi-trailer	Tractor-truck	Semi-trailer	Tractor-truck	Semi-trailer	Tractor-truck	Semi-trailer	Tractor-truck	Semi-trailer	Full trailer
Body type	Club coupe, 6-pass.	4-door, 6-pass.	Pickup	Stake	Van	Dump	Tractor	28-ft. van	Tractor	33-ft. van	Tractor	32-ft. van	Tractor	35-ft. van	Tractor	24-ft. van	24-ft. van
Price, f. o. b. factory (1954)	$1.790	$2,555	$1,562	$2.217	$5,263	$10,671	$5.631	$5,200	$7.150	$5,785	$10.868	$8,087	$16.346	$8,788	$18.312	$4,800	$6.800
Horsepower, N. A. C. C. (S. A. E.)	25.35	51.20	31.5	30.40	46.51	38.40	46.20	------	38.40	------	40.84	------	45.90	------	40.84	------	------
Horsepower, maximum brake	110	195	115	108	153	145	155	------	145	------	200	------	165	------	200	------	------
Weight:																	
Manufacturer's rated capacity..tons	------	------	½	1½	2¾	------	------	------	------	------	------	------	------	------	------	------	------
Chassis weight..pounds	------	------	------	3,730	5,477	12,200	------	------	------	------	------	------	------	------	------	------	------
Empty weight..do	2,982	3,805	3,266	5,488	7,675	17,000	7,915	7,300	9.620	7.470	10,050	9,000	17,015	9,500	11,300	6,585	9,560
Load capacity..do	------	------	1,534	7,012	11,325	23,000	14.085	10.700	13,380	19,530	12,950	18,000	18,985	16.500	13,700	8,415	22,440
Gross vehicle weight..do	3,882	4.705	4,800	12,500	19,000	40,000	22.000	18.000	23,000	27,000	23.000	27,000	36,000	26,000	25,000	15,000	32.000
Gross weight of combination..do	------	------	------	------	------	------	40,000	------	50,000	------	50,000	------	62,000	------	72,000	------	------
Length:																	
Wheelbase..inches	------	------	110	154	174	178–48½	147	------	142	------	140	------	145–50	------	(1)	(1)	(1)
Distance between tandem axles..do	------	------	------	------	------	48½	------	------	------	48½	------	48½	50	48½	------	------	------
Length of combination..feet	------	------	------	------	------	------	39	------	43	------	43	------	49	------	60	------	------
Tires:																	
Size	------	------	6.50 x 16	7.50 x 20	9.00 x 20	10.00 x 20	10.00 x 20	10.00 x 20	10.00 x 20	10.00 x 20	10.00 x 20	10.00 x 20	10.00 x 20	10.00 x 20	10.00 x 20	10.00 x 20	10.00 x 20
Ply rating	------	------	6	8	10	12	12	12	12	12	12	12	14	14	12	12	12
Front arrangement	------	------	Single	Single	Single	Single	Single	------	Single	------	Single	------	Single	------	Single	------	Dual
Rear arrangement	------	------	Single	Dual	Dual	Dual-tandem	Dual	Dual	Dual	Dual-tandem	Dual	Dual-tandem	Dual-tandem	Dual-tandem	Dual	Dual	Dual
Total annual travel:																	
Farm service..miles	------	------	6,000	5,500	------	------	------	------	------	------	------	------	------	------	------	------	------
Private operation..do	9,500	10,500	9.000	12.000	15,000	30,000	40,000	------	60,000	------	60,000	------	70,000	------	70,000	------	------
Contract carrier..do	------	------	------	20,000	25,000	------	40,000	------	60,000	------	60,000	------	70,000	------	70,000	------	------
Average miles per gallon:																	
Farm service	------	------	15.5	10.0	------	------	------	------	------	------	------	------	------	------	------	------	------
Private operation	16.5	14.5	15.0	9.0	6.5	4.0	5.0	------	4.0	------	6.0	------	4.7	------	4.3	------	------
Contract carrier	------	------	------	9.0	6.5	------	5.0	------	4.0	------	6.0	------	4.7	------	4.3	------	------
Annual motor-fuel consumption:																	
Farm service..gallons	------	------	387	550	------	------	------	------	------	------	------	------	------	------	------	------	------
Private operation..do	576	724	600	1,333	2,307	7,500	8,000	------	15,000	------	10,000	------	14,894	------	16,279	------	------
Contract carrier..do	------	------	------	2,222	3,846	------	8,000	------	15,000	------	10,000	------	14,894	------	16,279	------	------
Total revenue ton-miles (average load)	------	------	------	39,000	78.000	------	297,000	------	642,000	------	604,000	------	918,000	------	1,123,000	------	------
Average gross weight..pounds	------	------	------	9,345	13,904	------	30,086	------	38.482	------	39,168	------	52,755	------	59,525	------	------
Gross annual earnings	------	------	------	$8.200	$12,000	------	$24,000	------	$49,000	------	$47,000	------	$70,000	------	$79,000	------	------

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[1] Distances between axles on vehicle No. 11 are as follows: first and second, 114 inches; second and third, 221 inches; third and fourth, 98¾ inches; and fourth and fifth, 233 inches.

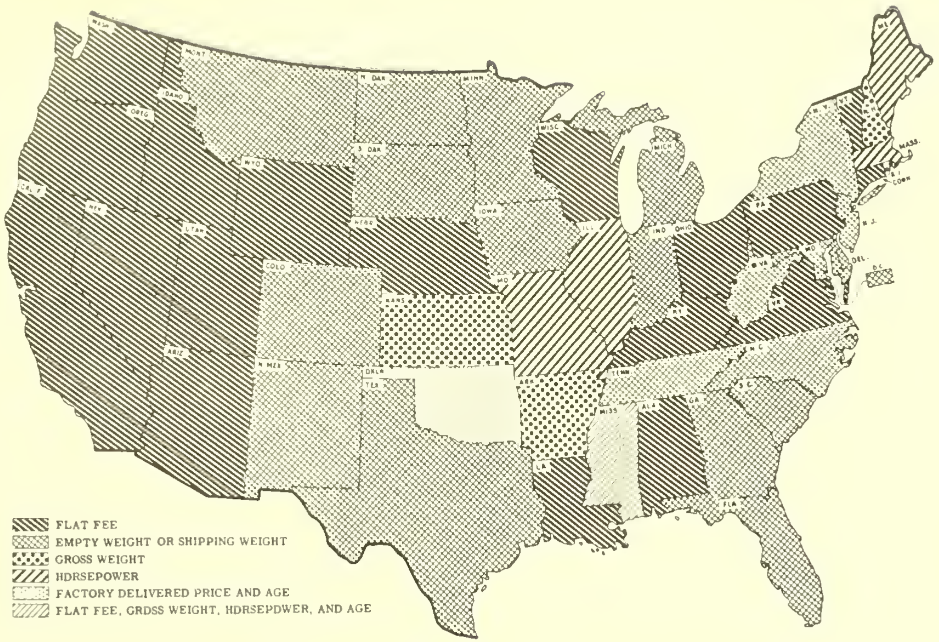

FLAT FEE
EMPTY WEIGHT OR SHIPPING WEIGHT
GROSS WEIGHT
HORSEPOWER
FACTORY DELIVERED PRICE AND AGE
FLAT FEE, GROSS WEIGHT, HORSEPOWER, AND AGE

Figure 1.—Passenger-car registration fee basis.

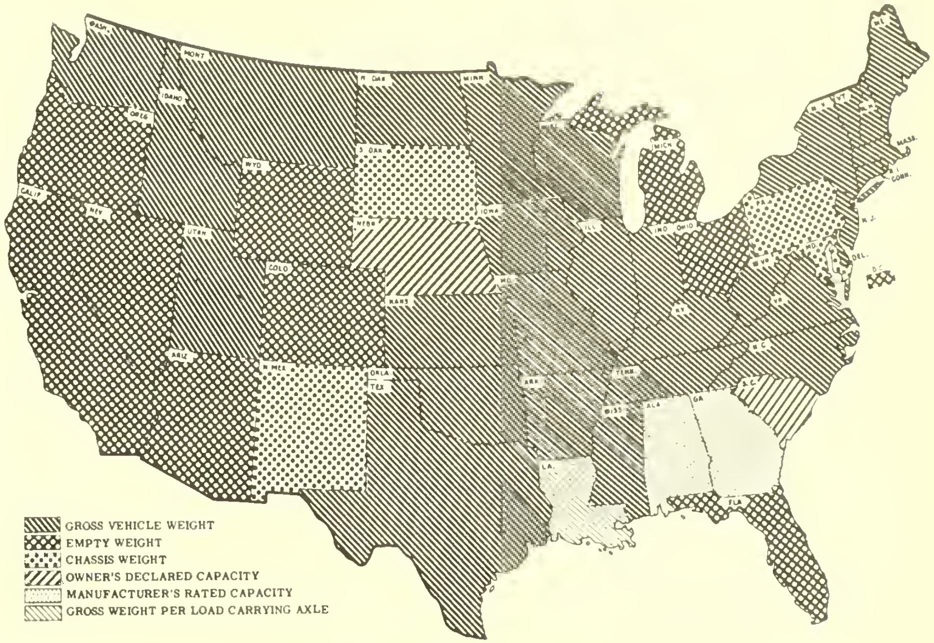

GROSS VEHICLE WEIGHT
EMPTY WEIGHT
CHASSIS WEIGHT
OWNER'S DECLARED CAPACITY
MANUFACTURER'S RATED CAPACITY
GROSS WEIGHT PER LOAD CARRYING AXLE

Figure 2.—Truck registration fee basis.

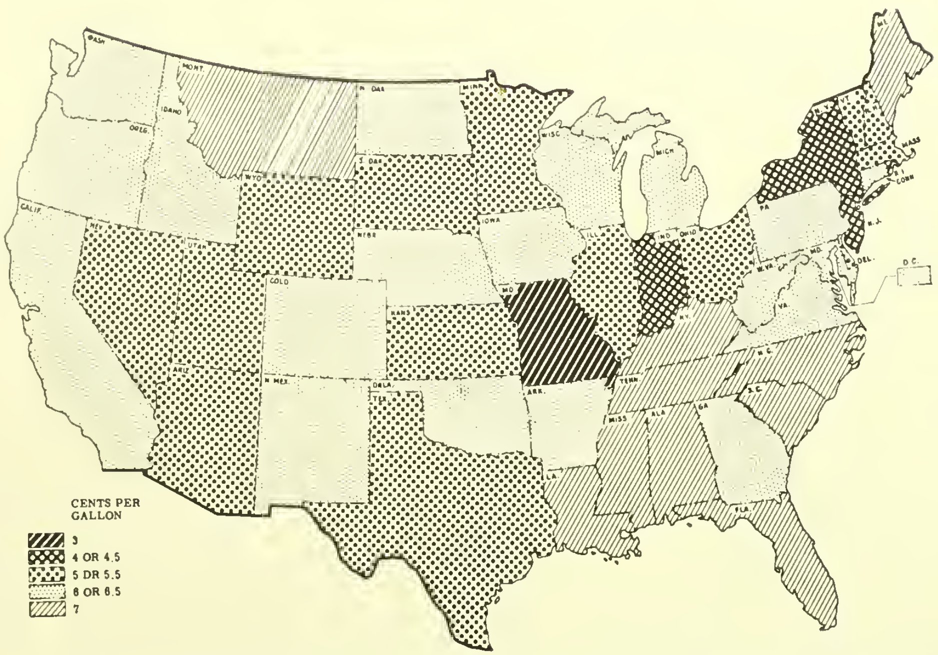


Figure 3.—State gasoline tax rates.

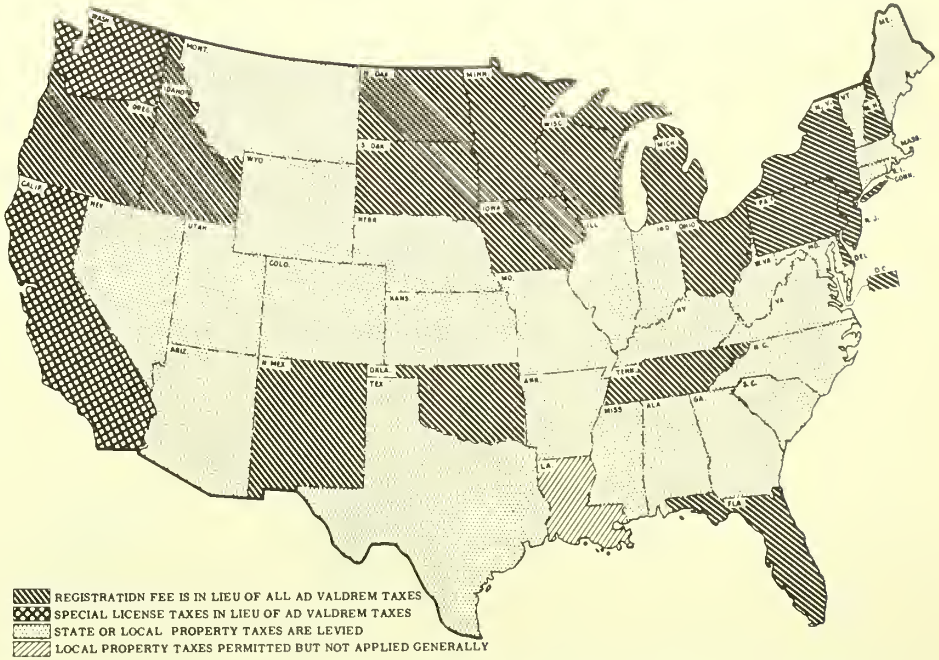

REGISTRATION FEE IS IN LIEU OF ALL AD VALOREM TAXES
SPECIAL LICENSE TAXES IN LIEU OF AD VALOREM TAXES
STATE OR LOCAL PROPERTY TAXES ARE LEVIED
LOCAL PROPERTY TAXES PERMITTED BUT NOT APPLIED GENERALLY

Figure 4.—Application of personal-property taxes to motor vehicles.

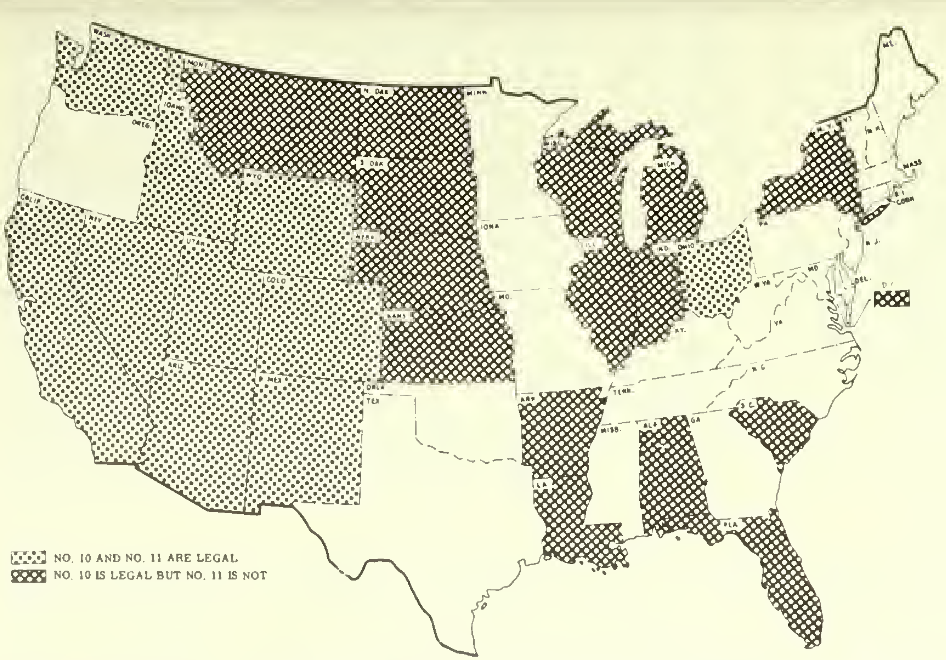

NO. 10 AND NO. 11 ARE LEGAL
NO. 10 IS LEGAL BUT NO. 11 IS NOT

Figure 5.—Legality of 62,000-pound tractor-semitrailer (No. 10) and 72,000-pound tractor-semitrailer and full trailer (No. 11).

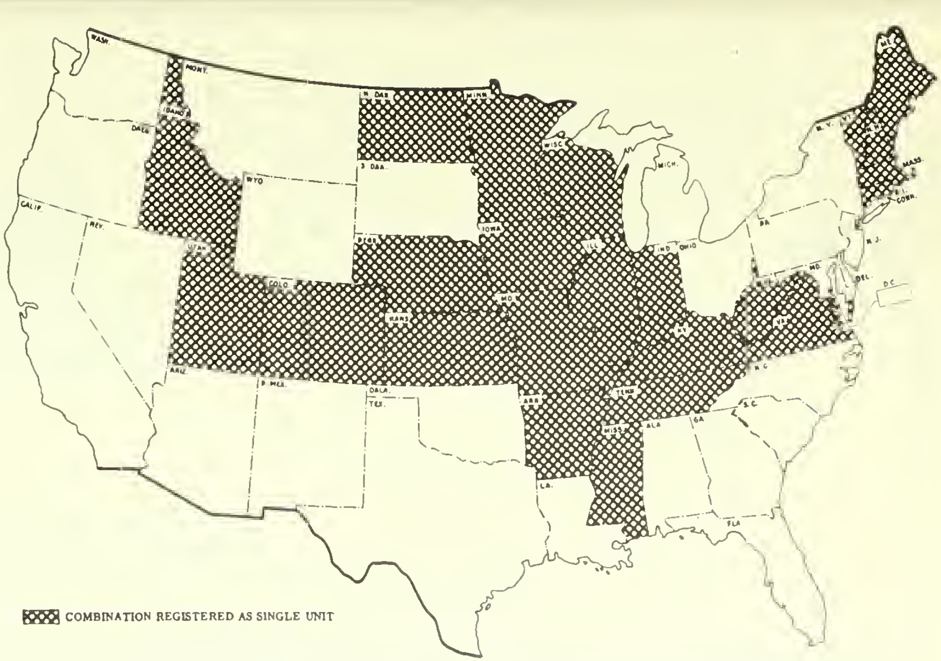


Figure 6.—Registration of tractor-semitrailer as a single unit.

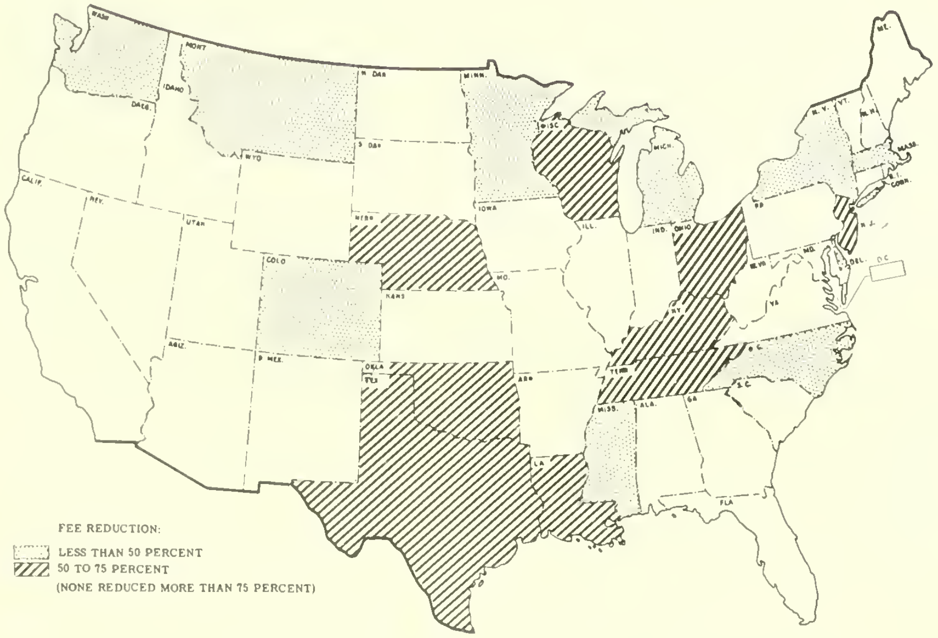


Figure 7.—Reduced registration fees for farm truck (No. 3, pickup truck).

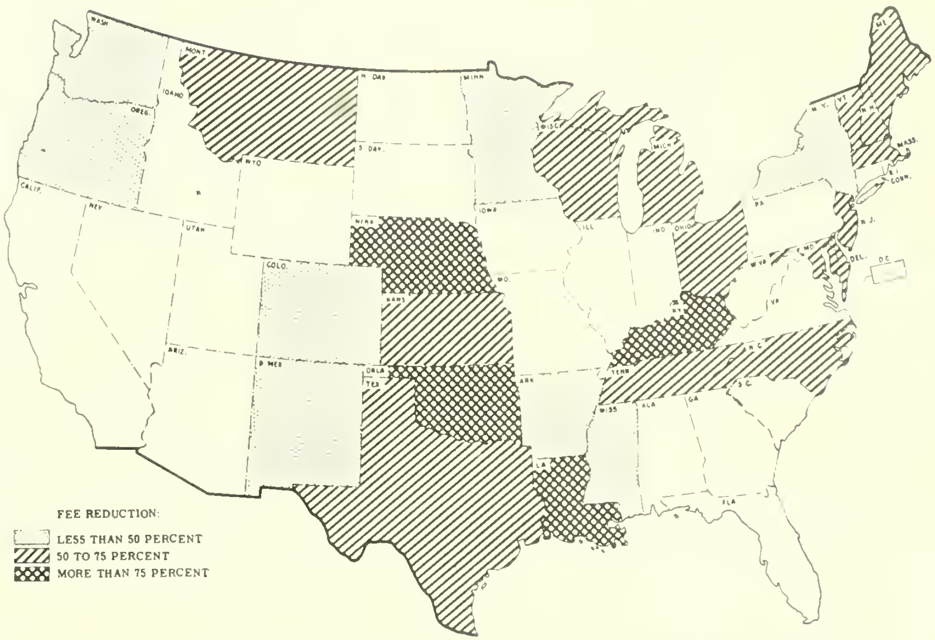


Figure 8.—Reduced registration fees for farm truck (No. 4, stake truck).

Table 3.—Average, low, and high road-user and property taxes on selected motor vehicles[1]

Vehicle and service	Average fee for all States					Lowest fee					Highest fee				
	Registration fee, etc.	Motor-fuel tax	Total road-user taxes	Property tax	Total	Registration fee, etc.	Motor-fuel tax	Total road-user taxes	Property tax	Total	Registration fee, etc.	Motor-fuel tax	Total road-user taxes	Property tax	Total
Passenger car:															
Lightweight (No. 1)	$13.43	$33.21	$46.64	$28.14	$61.57	$3.00	$17.28	$28.28	($5.00)	$38.04	$31.61	$40.32	$67.91	$52.07	$99.59
Mediumweight (No. 2)	17.71	41.74	59.45	40.63	81.01	3.00	21.72	40.20	(8.00)	46.20	42.50	50.68	87.25	74.35	132.41
Pickup (No. 3):															
Farm	15.37	22.31	37.68	17.56	46.99	3.00	11.61	24.35	(2.84)	30.09	45.00	27.09	68.22	45.38	75.86
Private	19.56	34.59	54.15	24.23	67.00	5.00	18.00	33.00	(5.00)	46.00	45.00	42.00	81.00	45.38	100.94
Stake truck (No. 4):															
Farm	33.10	31.71	64.81	24.66	77.89	5.00	16.50	36.50	(4.97)	48.50	86.00	38.50	113.50	64.42	128.94
Private	58.50	76.85	135.35	33.47	153.11	15.00	39.99	59.99	(8.00)	84.51	141.55	93.31	221.53	64.42	253.88
Contract	95.20	128.11	223.31	33.47	241.07	25.00	66.66	132.88	(8.00)	149.88	328.17	155.54	461.49	64.42	461.49
Van (No. 5):															
Private	112.56	133.01	245.57	72.28	283.92	44.44	69.21	142.28	(18.91)	183.28	275.00	161.49	401.49	153.11	403.92
Contract	171.39	221.75	393.14	72.28	431.49	59.00	115.38	212.84	(18.91)	244.84	505.67	269.22	736.43	153.11	736.43
Dump (No. 6):															
Private	319.87	431.25	751.12	137.43	825.56	100.00	225.00	325.00	(25.46)	383.38	872.50	525.00	1,247.50	310.45	1,247.50
Three-axle combination (No. 7):															
Private	333.51	461.22	794.74	157.13	878.11	76.00	240.00	449.00	(45.00)	522.00	867.10	560.00	1,347.10	298.10	1,470.91
Contract	425.52	461.22	886.75	157.13	970.12	134.00	240.00	454.00	(45.00)	522.00	1,034.33	560.00	1,514.33	298.10	1,514.33
Four-axle combination, gasoline (No. 8):															
Private	486.15	860.94	1,347.08	182.83	1,442.31	76.00	450.00	755.00	(55.00)	882.00	1,717.66	1,050.00	2,617.66	356.05	2,772.88
Contract	635.11	860.94	1,496.05	182.83	1,591.27	160.00	450.00	760.00	(55.00)	882.00	1,893.83	1,050.00	2,793.83	356.05	2,793.83
Four-axle combination, diesel (No. 9):															
Private	529.91	594.57	1,099.70	257.14	1,233.63	76.00	300.00	555.00	(55.00)	682.00	2,533.70	900.00	2,533.70	521.76	2,533.70
Contract	675.47	594.57	1,245.26	257.14	1,379.19	160.00	300.00	560.00	(55.00)	682.00	2,533.70	900.00	2,611.83	521.76	2,611.83
Five-axle combination, diesel (No. 10):															
Private	851.30	899.38	1,750.68	384.71	1,943.03	151.00	595.76	895.76	(87.96)	1,189.64	2,601.92	1,340.46	3,495.56	691.86	3,797.17
Contract	1,113.92	899.38	2,013.31	384.71	2,205.66	296.00	595.76	919.76	(87.96)	1,189.64	2,845.33	1,340.46	3,887.91	691.86	3,887.91
Five-axle combination, diesel (No. 11):															
Private	1,190.24	919.76	2,110.00	454.40	2,337.20	239.90	651.16	1,053.85	(358.94)	1,313.64	3,042.96	1,139.53	4,019.70	673.00	4,378.64
Contract	1,666.77	919.76	2,586.53	454.40	2,813.73	336.90	651.16	1,283.95	(358.94)	1,313.64	3,164.17	1,139.53	4,303.70	673.00	4,378.64

[1] In 22 States and the District of Columbia, personal-property taxes are not imposed. In one State diesel fuel is not taxed directly. The average fees given are for the States where the respective taxes are collected. The fees in parentheses are the lowest for the same States.

specified, would be permitted to register and operate. Some instances were found in which the vehicle or combination might be registered but could not be operated legally with a full load. Even though they could be registered and might, in some cases, be operated on the highways of the State with slightly smaller gross weights, all such vehicles were omitted from the study. Minor adjustments of axle spacing, however, were assumed in a few cases in order to include vehicles that otherwise were within the legal limits.

As shown in figure 6 (p. 41), tractor-semitrailer combinations are registered as single units in about half of the States and in the remainder, tractor-trucks and semitrailers are registered as separate units. Combinations are considered to be registered as single units where the fee for either the tractor or the semitrailer is based on the gross weight or capacity of the combination, even though in most of these States a relatively small fee is also levied on the other unit of the combination.

Specifications and Use Factors

In order to obtain the tax data on a uniform basis in all States, a detailed statement of the specifications of each of the 11 vehicles selected was submitted to the State authorities in the latter part of 1955 for determination of the specific fees and taxes that would be payable during a full year under laws existing January 1, 1956. Table 2 (p. 39) gives these specifications together with the assumed annual mileage traveled, motor fuel consumed, and other factors needed to compute the various taxes. The taxes to be reported were the State road-user taxes—on motor fuel, on vehicle registration, other annual vehicle fees closely related to registration, and the applicable motor-carrier taxes—and all direct personal-property taxes levied on motor vehicles at all levels of government. The property taxes for 1956 were estimated by the State authorities on the basis of 1955 property valuations and tax rates.

The same care that was taken in the selection of vehicles was taken in the selection of use factors. Average values are not available, however, for the mileages, fuel-consumption rates, and earnings. It was therefore necessary to assign these values arbitrarily, but special effort was made to select amounts that are reasonable. The objective was to supply specific factors to which rates could be applied uniformly in all States, and to present data reflecting these rates to permit valid comparisons between States, vehicles, and services. In order to avoid the complex situations that would have been encountered in computing taxes on vehicles in interstate operation, intrastate use was specified. Without this stipulation, the study would have been impractical. Obviously, few large combinations travel entirely in one State for a whole year. However, the stipulation of intrastate travel is permissible, even indispensable, when it is remembered that the purpose of the study is to compare tax rates rather than to select the most typical vehicles, or the most probable amount and kind of service or the most probable tax payments.

It was specified that the vehicles had been operated in the same State since purchased new in 1954. This made 1956 the third year of registration, thus excluding from the study such nonrecurring taxes as those on sales and titling, and removing the vehicles from the top property-tax brackets.

Another large group of taxes omitted are those imposed by the Federal Government on gasoline, oil, vehicles, and accessories and parts, which yielded over $2.9 billion in 1955. Although newly enacted Federal highway legislation now earmarks much of the revenue from these taxes, they were in the past identical in nature with taxes levied on tobacco, cosmetics, and so forth, and were collected on the products whether they were used for highway or nonhighway purposes. In any case, their exclusion has no effect on comparison between States of the data presented in this article since the levies are uniform throughout the Nation.

The data received from the States are presented in tables 4–13. In all of these tables (except table 13) the States are arranged in regional groupings that permit ready comparison of data for each State with those of its neighbors.

The national averages and extremes are shown in table 3. The averages are simple unweighted averages of the totals for all States where the vehicles are permitted. In the case of property taxes, they are averages reflecting only the States where such taxes are levied. Figure 9 gives a quick graphic comparison of the low, average, and high annual rates on four of the vehicles covered in the study. Figures 11–20 show graphically the principal groups of taxes, and permit quick comparisons between the elements reported, and between the States.

Property Taxes

In most States property taxes on motor vehicles have little or no direct relation to the use of highways, and the revenues from them are not available for highways. They are, however, so closely associated with registration fees in their application and their relation

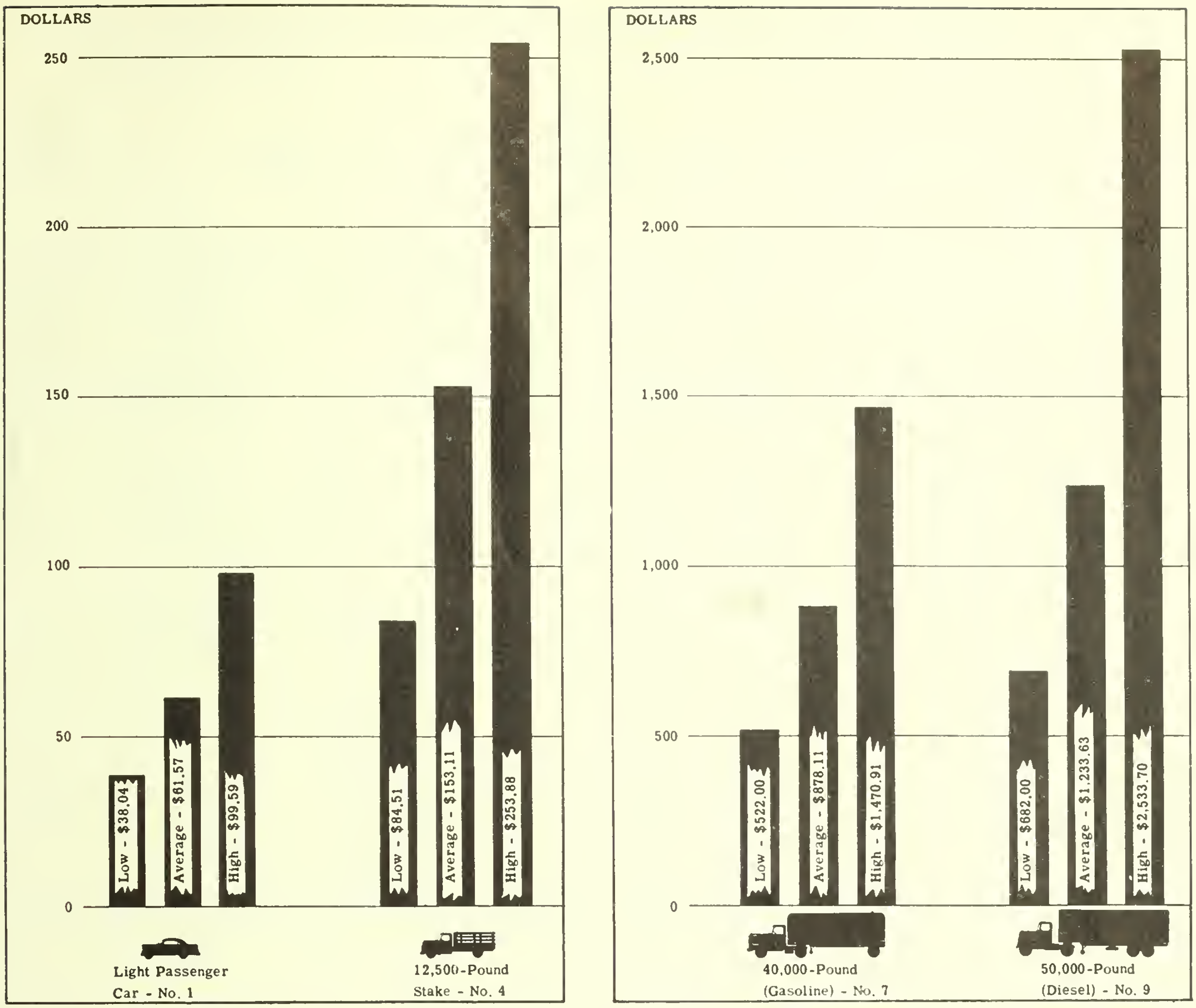


Figure 9.—Annual rates of low, high, and average taxes (road-user and property) on selected vehicles in private operation.

to costs of operation and comprise such a large portion of the total taxes paid on motor vehicles in some States, that their inclusion was necessary in order to obtain equitable comparisons. The registration fees for individual vehicles vary considerably among the States, and property taxes in some instances greatly exceed the registration fees paid; but when property taxes and registration fees are considered together, the differences among the States are smaller in most instances.

Figure 4 shows the States in which State or local property taxes are levied and the States in which registration fees are in lieu of ad valorem or property taxes. In this connection it should be noted that the registration fees of quite a few States are based, at least in part, on the age or valuation of the vehicle. Louisiana permits local property taxes to be levied in addition to the registration fee, but the application of such levies is not general in the State. The relation of property taxes to registration fees and total taxes on motor vehicles is shown in tables 4–13.

The valuation of motor vehicles for tax purposes is generally conceded to be much closer to market value than is the valuation of other personal property (household furnishings, for instance) subject to the same taxes. It is also probable that motor vehicles constitute a substantial portion of taxable personal property in many jurisdictions. Therefore, although this study is not directly concerned with personal-property taxes per se, the total tax burden on the motor vehicle must not be neglected either by those concerned solely with highway finance, or in research in broader fields of public finance.

The registration fee for the lightweight passenger car in Pennsylvania, as shown in table 4, is $10, more than twice as much as the $4.50 levied by Massachusetts. In Pennsylvania, however, the flat registration fee of $10, that applies to all automobiles, is in lieu of all other ad valorem taxes, whereas in Massachusetts vehicles are subject to property taxes in addition to registration fees. The addition of the $39.50 property tax to the $4.50 registration fee yields a total of $44, which is 4.4 times as much as the Pennsylvania registration fee, and far greater than the registration fee alone for any State. Oklahoma, Iowa, North Dakota, and Vermont, all with registration fees in excess of $25, appear at first glance to be exacting greater taxes on the lightweight automobile than other States. Yet in none of these States is there a property tax on motor vehicles, and when property taxes and registration fees are combined, the total in a great many States exceeds the fees of the four mentioned. Although only a few of the States are pointed out here as examples, many other significant comparisons can be made from the tables.

Another factor that demonstrates the close relation between registration fees and property taxes is the reduction of the registration fee in several States according to the age (and value) of the vehicle.

The property taxes that are given in tables 4–13, for all except the farm vehicles, include all taxes levied by the State, county, city, or other local governmental unit in which the vehicle is domiciled, and are the taxes that would be imposed on the vehicles in the capital city of the State. The property taxes given for farm vehicles include State, county, and district or other property taxes that would be collected in an average rural agricultural community in the State. In a few States where uniform statewide valuation and tax rates are in effect in all jurisdictions, there is no difference between property taxes on farm vehicles and those registered in the State capital.

Table 4.—Road-user and personal-property taxes on selected passenger vehicles (Nos. 1 and 2)

State	Lightweight passenger car (No. 1)							Mediumweight passenger car (No. 2)						
						Rank of State							Rank of State	
	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax
New England:														
Maine	$14.00	$22.38	------	$40.32	$76.70	8	10	$16.00	$31.94	------	$50.68	$98.62	12	13
New Hampshire	15.50	------	$16.11	28.80	60.41	25	6	19.50	------	$23.00	36.20	78.70	26	6
Vermont	26.00	------	------	31.68	57.68	28	7	26.00	------	------	39.82	65.82	38	16
Massachusetts	4.50	39.50	------	28.80	72.80	11	47	9.00	56.50	------	36.20	101.70	9	44
Rhode Island	12.00	24.18	------	23.04	59.22	27	43	16.00	40.30	------	28.96	85.26	18	45
Connecticut	8.00	33.82	------	34.56	76.38	9	31	8.00	49.40	------	43.44	100.84	10	36
Middle Atlantic:														
New York	15.00	------	------	23.04	38.04	49	42	19.75	------	------	28.96	48.71	47	40
New Jersey	15.00	------	------	23.04	38.04	48	41	25.00	------	------	28.96	53.96	43	30
Pennsylvania	10.00	------	------	34.56	44.56	44	26	10.00	------	------	43.44	53.44	46	33
Delaware	10.00	------	------	28.80	38.80	47	40	10.00	------	------	36.20	46.20	49	43
Maryland	10.00	5.00	------	34.56	49.56	40	24	15.00	8.00	------	43.44	66.44	37	22
District of Columbia	22.00	------	1.00	34.56	57.56	29	8	32.00	------	1.00	43.44	76.44	28	7
West Virginia	17.00	11.64	------	34.56	63.20	21	15	22.40	14.55	------	43.44	80.39	25	15
Southeastern:														
Virginia	10.00	27.28	------	34.56	71.84	13	27	10.00	51.48	------	43.44	104.92	8	34
North Carolina	10.00	28.04	------	40.32	78.36	7	20	12.00	42.78	------	50.68	105.46	7	18
South Carolina	4.00	41.14	------	40.32	85.46	4	28	6.00	54.86	------	50.68	111.54	4	26
Georgia	3.00	26.91	------	37.44	67.35	15	35	10.00	35.53	------	47.06	92.59	14	24
Florida	15.00	------	.25	40.32	55.57	32	9	20.00	------	.25	50.68	70.93	34	9
Kentucky	4.50	35.82	.50	40.32	81.14	5	22	4.50	50.75	.50	50.68	106.43	5	27
Tennessee	9.50	------	------	40.32	49.82	39	21	13.00	------	------	50.68	63.68	39	17
Alabama	3.00	15.41	.75	40.32	59.48	26	29	3.00	18.76	.75	50.68	73.19	32	29
Mississippi	12.48	45.42	------	40.32	98.22	2	11	16.57	65.16	------	50.68	132.41	1	12
East Central:														
Ohio	10.25	------	------	28.80	39.05	46	39	10.25	------	------	36.20	46.45	48	42
Indiana	11.00	52.07	------	23.04	86.11	3	44	12.00	74.35	------	28.96	115.31	3	48
Illinois	10.50	21.81	------	28.80	61.11	24	38	22.00	34.90	------	36.20	93.10	13	23
Michigan	10.50	------	------	34.56	45.06	42	23	13.30	------	------	43.44	56.74	42	25
Wisconsin	16.00	------	------	34.56	50.56	38	17	16.00	------	------	43.44	59.44	41	21
Minnesota	23.20	------	.25	28.80	52.25	35	13	39.00	------	.25	36.20	75.45	29	8
Iowa	29.00	------	------	34.56	63.56	19	4	38.00	------	------	43.44	81.44	24	5
Missouri	11.00	19.85	------	17.28	48.13	41	49	25.00	28.02	------	21.72	74.74	30	41
Southwestern:														
Arkansas	13.00	15.19	------	37.44	65.63	16	18	19.00	21.56	------	47.06	87.62	16	14
Louisiana	3.00	------	------	40.32	43.32	45	30	3.00	------	------	50.68	53.68	44	31
Oklahoma	30.47	------	------	37.44	67.91	14	1	40.19	------	------	47.06	87.25	17	1
Texas	11.16	16.93	------	28.80	56.89	31	37	18.74	23.61	------	36.20	78.55	27	28
West Central:														
North Dakota	27.50	------	------	34.56	62.06	23	5	38.50	------	------	43.44	81.94	23	4
South Dakota	22.00	------	------	28.80	50.80	37	16	33.00	------	------	36.20	69.20	36	11
Nebraska	8.00	29.37	------	34.56	71.93	12	32	8.00	40.57	------	43.44	92.01	15	37
Kansas	13.15	34.25	------	28.80	76.20	10	33	15.95	48.64	------	36.20	100.79	11	35
Mountain:														
Montana	10.00	49.27	------	40.32	99.59	1	19	10.00	70.33	------	50.68	131.01	2	20
Idaho	17.50	------	------	34.56	52.06	36	14	17.50	------	------	43.44	60.94	40	19
Wyoming	5.00	21.48	------	28.80	55.28	33	46	5.00	30.66	------	36.20	71.86	33	47
Colorado	5.60	21.48	1.00	34.56	62.64	22	34	6.95	30.66	1.00	43.44	82.05	22	38
New Mexico	18.00	------	------	34.56	52.56	34	12	26.00	------	------	43.44	69.44	35	10
Arizona	4.00	24.20	------	28.80	57.00	30	48	4.00	34.40	------	36.20	74.60	31	49
Utah	5.00	29.56	------	28.80	63.36	20	45	5.00	41.82	------	36.20	83.02	20	46
Nevada	5.50	39.60	------	34.56	79.66	6	36	5.50	56.85	------	43.44	105.79	6	39
Pacific:														
Washington	5.00	------	21.40	37.44	63.84	18	3	5.00	------	30.40	47.06	82.46	21	3
Oregon	10.00	------	------	34.56	44.56	43	25	10.00	------	------	43.44	53.44	45	32
California	8.00	------	23.00	34.56	65.56	17	2	8.00	------	32.00	43.44	83.44	19	2

In two States—California and Washington—property taxes on motor vehicles have been replaced by "in lieu" taxes. These taxes have many characteristics of property taxes, but since neither the levies nor the distribution of their proceeds are directly related to the jurisdiction in which they were collected, they have been classified as road-user imposts. In both of these instances the State courts have ruled them not to be property taxes.

Types of Service

Distinctly different tax rates are in effect in most States for commercial vehicles operated privately (not for hire), as contract carriers (for hire), and in farm service. Information was therefore obtained and tabulated separately for all vehicles in private operation, for the pickup and stake trucks (Nos. 3–4) in farm service, and for the stake and van trucks (Nos. 4–5) and all combinations (Nos. 7–11) in contract carrier service.

Vehicles in private operation are those used solely for carrying goods owned or sold by the vehicle owner, with no direct transportation charge.

Contract carriers are those employed in hauling goods for others, with a direct transportation charge, at times and to destinations required by the jobs for which they are hired. Operation of contract carriers is usually subject to State franchise and regulation. Common carriers, which operate for hire over established routes and on fixed schedules, are not included in the study because of their more complex tax schedules and forms of regulation, and the fact that their operation is predominantly interstate.

The farm-service classification includes vehicles registered at a reduced fee without restriction as to the highways they may use. Provision for reduced fees for farm trucks, in various forms, is in effect in 27 States. (Farm vehicles registered by some States at a nominal charge imposed as the sole registration fee and licensed for restricted operation on or in the immediate vicinity of the farm are not included in this study.)

Farm Service

Provision for registration of farm trucks at half the regular fee for private trucks is common but, as shown in tables 5 and 6, some States do not allow reductions that great, and some allow considerably more. The reductions for farm vehicles vary not only among the States but also vary somewhat for vehicles of different capacities, as may be seen from a comparison of figures 7 and 8 (p. 41). Large numbers of farm trucks are in the lighter weight groups, and what may at first glance appear to be relatively minor concessions in registration fees can, in fact, be of major importance from a revenue standpoint, particularly in predominantly agricultural States. In four States, as indicated in figure 8, the 12,500-pound stake truck (No. 4), if classified as a farm truck, is allowed a reduction of more than 75 percent in registration fee, in 15 States the reduction is from 50 to 75 percent, and only 8 of the States allowing a reduction place it below 50 percent.

A comparison of the regular and farm registration fees for vehicle No. 4 in each State is shown in figure 10. In Nebraska the regular registration fee for vehicle No. 4 is $90, but a farmer can register the same truck for $12. In Kentucky, farm trucks of 22,000 pounds or less can be registered for $4.50 but

Table 5.—Road-user and personal-property taxes on a pickup truck, 4,800 pounds gross vehicle weight (No. 3)

State	Farm service							Private operation						
	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Rank of State		Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Rank of State	
						Total fees and taxes	Total, excluding property tax						Total fees and taxes	Total, excluding property tax
New England:														
Maine	$15.00	$19.53	------	$27.09	$61.62	7	11	$15.00	$19.53	------	$42.00	$76.53	11	17
New Hampshire	19.50	------	$14.06	19.35	52.91	12	3	19.50	------	$14.06	30.00	63.56	30	10
Vermont	26.00	------	------	21.28	47.28	24	7	26.00	------	------	33.00	59.00	36	14
Massachusetts	12.00	34.50	------	19.35	65.85	4	37	15.00	34.50	------	30.00	79.50	9	42
Rhode Island	18.00	16.70	------	15.48	50.18	17	30	18.00	25.89	------	24.00	67.89	20	45
Connecticut	14.40	26.80	------	23.22	64.42	5	20	14.40	30.40	------	36.00	80.80	8	31
Middle Atlantic:														
New York	17.50	------	------	15.48	32.98	45	34	25.00	------	------	24.00	49.00	47	34
New Jersey	15.00	------	1.00	15.48	31.48	47	36	30.00	------	1.00	24.00	55.00	39	21
Pennsylvania	16.50	------	------	23.22	39.72	36	17	16.50	------	------	36.00	52.50	42	25
Delaware	11.50	------	------	19.35	30.85	48	40	17.50	------	------	30.00	47.50	48	38
Maryland	12.00	5.00	------	23.22	40.22	33	26	12.00	5.00	------	36.00	53.00	41	35
District of Columbia	44.00	------	1.00	23.22	68.22	2	1	44.00	------	1.00	36.00	81.00	7	1
West Virginia	20.00	7.56	------	23.22	50.78	16	9	20.00	10.18	------	36.00	66.18	26	19
Southeastern:														
Virginia	12.00	13.20	------	23.22	48.42	21	27	12.00	20.62	------	36.00	68.62	17	37
North Carolina	10.00	10.14	------	27.09	47.23	25	22	20.00	20.29	------	42.00	82.29	6	11
South Carolina	6.00	12.15	------	27.09	45.24	28	33	6.00	28.48	------	42.00	76.48	12	36
Georgia	5.00	22.33	------	25.16	52.49	13	42	5.00	22.33	------	39.00	66.33	25	43
Florida	24.75	------	.25	27.09	52.09	15	4	24.75	------	.25	42.00	67.00	21	5
Kentucky	4.50	16.00	.50	27.09	48.09	23	35	10.00	43.78	.50	42.00	96.28	3	24
Tennessee	9.50	------	------	27.09	36.59	40	24	25.00	------	------	42.00	67.00	22	6
Alabama	15.00	9.66	.50	27.09	52.25	14	10	15.00	15.41	.50	42.00	72.91	13	16
Mississippi	7.00	12.99	------	27.09	47.08	26	29	10.00	45.37	------	42.00	97.37	2	28
East Central:														
Ohio	17.35	------	------	19.35	36.70	39	23	36.95	------	------	30.00	66.95	23	7
Indiana	15.00	45.38	------	15.48	75.86	1	41	15.00	45.38	------	24.00	84.38	5	47
Illinois	20.00	13.82	------	19.35	53.17	11	18	20.00	18.54	------	30.00	68.54	18	32
Michigan	16.50	------	------	23.22	39.72	35	16	29.70	------	------	36.00	65.70	27	9
Wisconsin	10.00	------	------	23.22	33.22	44	32	35.00	------	------	36.00	71.00	15	3
Minnesota	20.40	------	.25	19.35	40.00	34	15	25.00	------	.25	30.00	55.25	38	20
Iowa	25.00	------	------	23.22	48.22	22	6	25.00	------	------	36.00	61.00	33	12
Missouri	15.00	9.68	------	11.61	36.29	41	48	15.00	18.68	------	18.00	51.68	46	49
Southwestern:														
Arkansas	12.00	8.25	------	25.16	45.41	27	21	12.00	12.25	------	39.00	63.25	31	30
Louisiana	3.00	------	------	27.09	30.09	49	43	10.00	------	------	42.00	52.00	44	27
Oklahoma	10.10	------	------	25.16	35.26	42	25	20.50	------	------	39.00	59.50	35	13
Texas	9.60	2.84	------	19.35	31.79	46	45	19.20	14.41	------	30.00	63.61	29	33
West Central:														
North Dakota	21.00	------	------	23.22	44.22	29	8	21.00	------	------	36.00	57.00	37	18
South Dakota	22.50	------	------	19.35	41.85	31	12	22.50	------	------	30.00	52.50	43	26
Nebraska	8.00	7.78	------	23.22	39.00	38	39	17.50	17.14	------	36.00	70.64	16	23
Kansas	10.00	28.30	------	19.35	57.65	8	44	10.00	31.51	------	30.00	71.51	14	46
Mountain:														
Montana	14.00	26.52	------	27.09	67.61	3	13	16.00	42.94	------	42.00	100.94	1	15
Idaho	17.50	------	------	23.22	40.72	32	14	17.50	------	------	36.00	53.50	40	22
Wyoming	5.00	18.74	------	19.35	43.09	30	49	5.00	18.74	7.50	30.00	61.24	32	44
Colorado	7.00	18.74	1.00	23.22	49.96	18	38	8.75	18.74	1.00	36.00	64.49	28	40
New Mexico	16.00	------	------	23.22	39.22	37	19	16.00	------	------	36.00	52.00	45	29
Arizona	15.55	21.00	------	19.35	55.90	9	28	15.55	21.00	------	30.00	66.55	24	41
Utah	7.50	23.07	------	19.35	49.92	19	47	7.50	23.07	------	30.00	60.57	34	48
Nevada	5.50	25.75	------	23.22	54.47	10	46	5.50	25.75	25.00	36.00	92.25	4	8
Pacific:														
Washington	9.75	------	14.95	25.16	49.86	20	5	14.50	------	14.95	39.00	68.45	19	4
Oregon	10.00	------	------	23.22	33.22	43	31	10.00	------	------	36.00	46.00	49	39
California	22.00	------	19.00	23.22	64.22	6	2	22.00	------	19.00	36.00	77.00	10	2

a $134 registration fee is required for a 22,000-pound vehicle in private use. In 1954, more than 54 percent of Nebraska's trucks and in Kentucky, over 39 percent, were registered at reduced farm rates. Similar situations exist in other States but in most the reductions are not as great. In New Hampshire, for example, a farm truck of less than 16,000 pounds may be registered at a flat fee of $25, but since this is greater than the regular fee of $19.50 for a pickup truck having a gross weight of 4,800 pounds, the farm rate is of no advantage to the owner of the pickup. Consequently, only 5 percent of the total truck registrations in New Hampshire are in the farm classification.

In 1954, 20 States were able to report farm truck registrations separately. In 3 of the States that did so, farm trucks constituted more than half of all trucks registered and farm trucks exceeded 30 percent of all trucks in 13 States. In considering the equity of special reduced rates for farm trucks, it should be remembered that farm trucks probably average fewer miles per year than other trucks. The reductions are not as great on a cents-per-mile basis as may appear from the comparisons in tables 5 and 6 and in figures 7 and 8.

Carrier Taxes

In consideration of motor-carrier service, the gross receipts taxes and other levies that are imposed on general business, and not limited to motor carriers, have been eliminated. Fees for authority to operate, and other fees paid only at the time a carrier begins to haul goods for hire, have also been excluded.

The point at which a road-user tax becomes a "carrier tax" is becoming ever more difficult to determine. Care should be taken in comparing the carrier taxes of the several States. Many of the States have special registration classes for vehicles used in contract (for hire) service. The fees in the special registration classes, in most cases, are substantially greater than the registration fees for vehicles in private (not-for-hire) service. The difference, while technically classed as part of the registration fee, is in reality a special carrier fee. There is, of course, the justifying factor that vehicles in carrier service usually operate a much greater mileage than those in private use, thus reducing the per-mile cost of their registration fees.

The majority of trucks are used for the ordinary private service of carrying goods owned or sold by the vehicle owner, and for which there is no direct transportation charge. Even these trucks, however, are subject to mileage or other carrier taxes under some conditions, and the variations in the tax structures of the States made it necessary in a few instances to make arbitrary decisions on the inclusion or exclusion of these special levies.

The general policy followed was to include only the taxes that must be paid on almost all of the vehicles in a given group. For example, the laws of some States are so written that practically all vehicles of more than 1½ tons (manufacturer's rated capacity) must pay certain mileage or compensatory taxes, whether in private or contract operation. In the same States many vehicles of 1½-ton capacity or less are not subject to the tax. In such cases the tax has been shown only for the heavier vehicles. It should be remembered that the purpose of the study is to present a comparison of tax rates and

Table 6.—Road-user and personal-property taxes on a stake truck, 12,500 pounds gross vehicle weight (No. 4)

State	Farm service							Private operation								Contract carrier								
						Rank of State								Rank of State									Rank of State	
	Registration fee	Property tax	Other taxes and fees	Gasoline tax	Total	Total fees and taxes	Total, excl. prop. tax	Registration fee	Property tax	Other taxes and fees	Mileage or ton-mile tax	Gasoline tax	Total	Total fees and taxes	Total, excl. prop. tax	Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Mileage or ton-mile tax	Gasoline tax	Total	Total fees and taxes	Total, excl. prop. tax
New England:																								
Maine	$30.00	$27.71	----	$38.50	$96.21	8	14	$60.00	$27.71	----	----	$93.31	$181.02	11	14	$60.00	$27.71	----	$30.00	----	$155.54	$273.25	15	16
New Hampshire	25.00	----	$19.95	27.50	72.45	27	9	75.00	----	$19.95	----	66.65	161.60	17	8	75.00	----	$19.95	2.00	----	111.10	208.05	36	27
Vermont	32.00	----	----	30.25	62.25	38	24	118.75	----	----	----	73.32	192.07	6	4	118.75	----	----	----	----	122.21	240.96	21	19
Massachusetts	12.00	49.00	----	27.50	88.50	14	48	39.00	49.00	----	----	66.65	154.65	19	39	39.00	49.00	----	15.00	----	111.10	214.10	30	40
Rhode Island	39.00	22.20	----	22.00	83.20	17	27	39.00	34.41	----	----	53.32	126.73	42	47	39.00	34.41	----	5.00	----	88.88	167.29	43	49
Connecticut	37.50	48.24	----	33.00	118.74	4	12	37.50	54.72	----	----	79.98	172.20	14	32	37.50	54.72	----	5.00	----	133.32	230.54	23	37
Middle Atlantic:																								
New York	43.75	----	----	22.00	65.75	33	19	62.50	----	----	----	53.32	115.82	45	36	62.50	----	----	----	----	88.88	151.38	48	44
New Jersey	30.00	----	1.00	22.00	53.00	47	37	60.00	----	1.00	----	53.32	114.32	46	37	60.00	----	1.00	----	----	88.88	149.88	49	45
Pennsylvania	55.00	----	----	33.00	88.00	15	7	55.00	----	----	----	79.98	134.98	34	24	55.00	----	----	----	----	133.32	188.32	39	32
Delaware	26.00	----	----	27.50	53.50	45	36	52.00	----	----	----	66.65	118.65	43	30	52.00	----	----	----	----	111.10	163.10	47	42
Maryland	13.00	7.00	----	33.00	53.00	46	43	25.00	8.00	----	----	79.98	112.98	47	40	25.00	8.00	----	----	----	133.32	166.32	44	43
District of Columbia	60.00	----	1.00	33.00	94.00	10	6	60.00	----	1.00	----	79.98	140.98	29	20	60.00	----	1.00	----	----	133.32	194.32	38	31
West Virginia	38.00	11.88	----	33.00	82.88	18	11	38.00	16.00	----	----	79.98	133.98	36	31	38.00	16.00	----	14.50	----	133.32	201.82	37	34
Southeastern:																								
Virginia	19.50	17.60	----	33.00	70.10	28	39	19.50	29.26	----	----	79.98	128.74	39	44	19.50	29.26	----	164.00	----	133.32	346.08	3	3
North Carolina	31.25	12.21	----	38.50	81.96	19	13	62.50	24.43	----	----	93.31	180.24	12	10	125.00	24.43	----	----	----	155.54	304.97	8	8
South Carolina	66.00	17.55	----	38.50	122.05	2	3	66.00	41.14	----	----	93.31	200.45	4	9	66.00	41.14	----	60.00	----	155.54	322.68	4	7
Georgia	15.00	26.39	----	35.75	77.14	22	40	15.00	26.39	----	----	86.64	128.03	41	43	30.00	26.39	----	25.00	----	144.43	225.82	24	29
Florida	60.50	----	.25	38.50	99.25	7	5	60.50	----	.25	----	93.31	154.06	20	12	60.50	----	.25	.50	$100.00	155.54	316.79	5	4
Kentucky	4.50	22.00	.50	38.50	65.50	34	45	32.00	43.78	.50	----	93.31	169.59	15	28	32.00	43.78	.50	82.00	----	155.54	313.82	6	12
Tennessee	22.50	----	----	38.50	61.00	40	28	45.00	----	----	----	93.31	138.31	32	23	70.00	----	----	12.50	----	155.54	238.04	22	21
Alabama	22.50	11.76	.50	38.50	73.26	25	25	22.50	18.76	.50	----	93.31	135.07	33	35	22.50	18.76	.50	----	100.00	155.54	297.30	11	9
Mississippi	21.40	16.76	----	38.50	76.66	23	32	37.00	55.97	----	----	93.31	186.28	7	26	73.00	55.97	----	13.00	----	155.51	297.51	10	18
East Central:																								
Ohio	35.75	----	----	27.50	63.25	35	20	84.25	----	----	----	66.65	150.90	23	16	84.25	----	----	20.00	----	111.10	215.35	29	24
Indiana	35.00	64.42	----	22.00	121.42	3	35	35.00	64.42	----	----	53.32	152.74	22	48	35.00	64.42	----	24.00	----	88.88	212.30	32	46
Illinois	86.00	15.44	----	27.50	128.94	1	1	86.00	20.72	----	----	66.65	173.37	13	15	86.00	20.72	----	----	----	111.10	217.82	27	30
Michigan	27.50	----	----	33.00	60.50	41	29	60.50	----	----	----	79.98	140.48	31	22	60.50	----	----	----	30.00	133.32	223.82	26	23
Wisconsin	35.00	----	----	33.00	68.00	30	16	140.00	----	----	----	79.98	219.98	2	2	140.00	----	----	10.00	----	133.32	283.32	14	6
Minnesota	33.60	----	.25	27.50	61.35	39	26	50.00	----	.25	----	66.65	116.90	44	34	50.00	----	.25	7.50	----	111.10	168.85	42	39
Iowa	70.00	----	----	33.00	103.00	6	4	70.00	----	----	----	79.98	149.98	25	17	70.00	----	----	5.00	----	133.32	208.32	34	26
Missouri	20.00	12.70	----	16.50	49.20	48	49	20.00	24.52	----	----	39.99	84.51	49	49	50.00	24.52	----	25.00	----	66.66	166.18	45	48
Southwestern:																								
Arkansas	36.00	15.73	----	35.75	87.48	16	10	42.00	21.83	----	----	86.64	150.47	24	27	42.00	21.83	----	----	----	144.43	208.26	35	33
Louisiana	10.00	----	----	38.50	48.50	49	41	60.00	----	----	----	93.31	153.31	21	13	120.00	----	----	10.00	----	155.54	285.54	13	5
Oklahoma	22.90	----	----	35.75	58.65	44	34	95.50	----	----	----	86.65	182.15	10	6	95.50	----	----	2.25	----	144.43	242.18	20	17
Texas	40.62	4.97	----	27.50	73.09	26	15	81.25	21.39	----	----	66.65	169.29	16	18	81.25	21.39	----	11.00	----	111.10	224.74	25	28
West Central:																								
North Dakota	33.75	----	----	33.00	66.75	32	18	33.75	----	28.00	----	79.98	141.73	28	19	33.75	----	28.00	55.00	----	133.32	250.07	19	15
South Dakota	35.00	----	----	27.50	62.50	37	22	35.00	----	95.00	----	66.65	196.65	5	3	35.00	----	95.00	10.00	----	111.10	251.10	18	14
Nebraska	12.00	14.76	----	33.00	59.76	43	44	90.00	32.52	----	----	79.98	202.50	3	7	90.00	32.52	----	15.00	----	133.32	270.84	16	20
Kansas	15.00	37.12	----	27.50	79.62	20	46	30.00	46.24	----	----	66.65	142.89	27	46	50.00	46.24	----	10.00	----	111.10	217.34	28	38
Mountain:																								
Montana	14.00	37.74	----	38.50	90.24	12	38	28.00	61.11	----	----	93.31	182.42	9	29	28.00	61.11	----	51.00	----	155.54	295.65	12	22
Idaho	30.00	----	----	33.00	63.00	36	21	30.00	----	----	----	79.98	109.98	48	38	30.00	----	----	----	----	133.32	163.32	46	41
Wyoming	15.00	26.60	----	27.50	69.10	29	47	15.00	26.60	2.50	$32.93	66.65	143.68	26	33	15.00	26.60	----	2.50	54.88	111.10	210.08	33	36
Colorado	13.75	26.60	1.00	33.00	74.35	24	42	22.80	26.60	1.00	----	79.98	130.38	38	41	17.50	26.60	1.00	----	121.90	133.32	300.32	9	11
New Mexico	34.13	----	----	33.00	67.13	31	17	51.20	----	----	----	79.98	131.18	37	25	51.20	----	----	----	----	133.32	184.52	40	35
Arizona	31.50	30.00	----	27.50	89.00	13	33	31.50	30.00	----	----	66.65	128.15	40	45	31.50	30.00	----	205.00	----	111.10	377.60	2	2
Utah	35.00	32.45	----	27.50	94.95	9	23	35.00	32.45	----	----	66.65	134.10	35	42	35.00	32.45	----	----	----	111.10	178.55	41	47
Nevada	27.50	32.35	----	33.00	92.85	11	30	27.50	32.35	114.05	----	79.98	253.88	1	1	27.50	32.35	----	114.05	----	133.32	307.22	7	10
Pacific:																								
Washington	17.50	----	24.00	35.75	77.25	21	8	30.00	----	24.00	----	86.64	140.64	30	21	30.00	----	24.00	15.00	----	144.43	213.43	31	25
Oregon	27.50	----	----	33.00	60.50	42	31	38.50	----	----	66.00	79.98	184.48	8	5	38.50	----	----	90.00	----	133.32	261.82	17	13
California	48.00	----	27.00	33.00	108.00	5	2	48.00	----	27.00	----	79.98	154.98	18	11	48.00	----	27.00	253.17	----	133.32	461.49	1	1

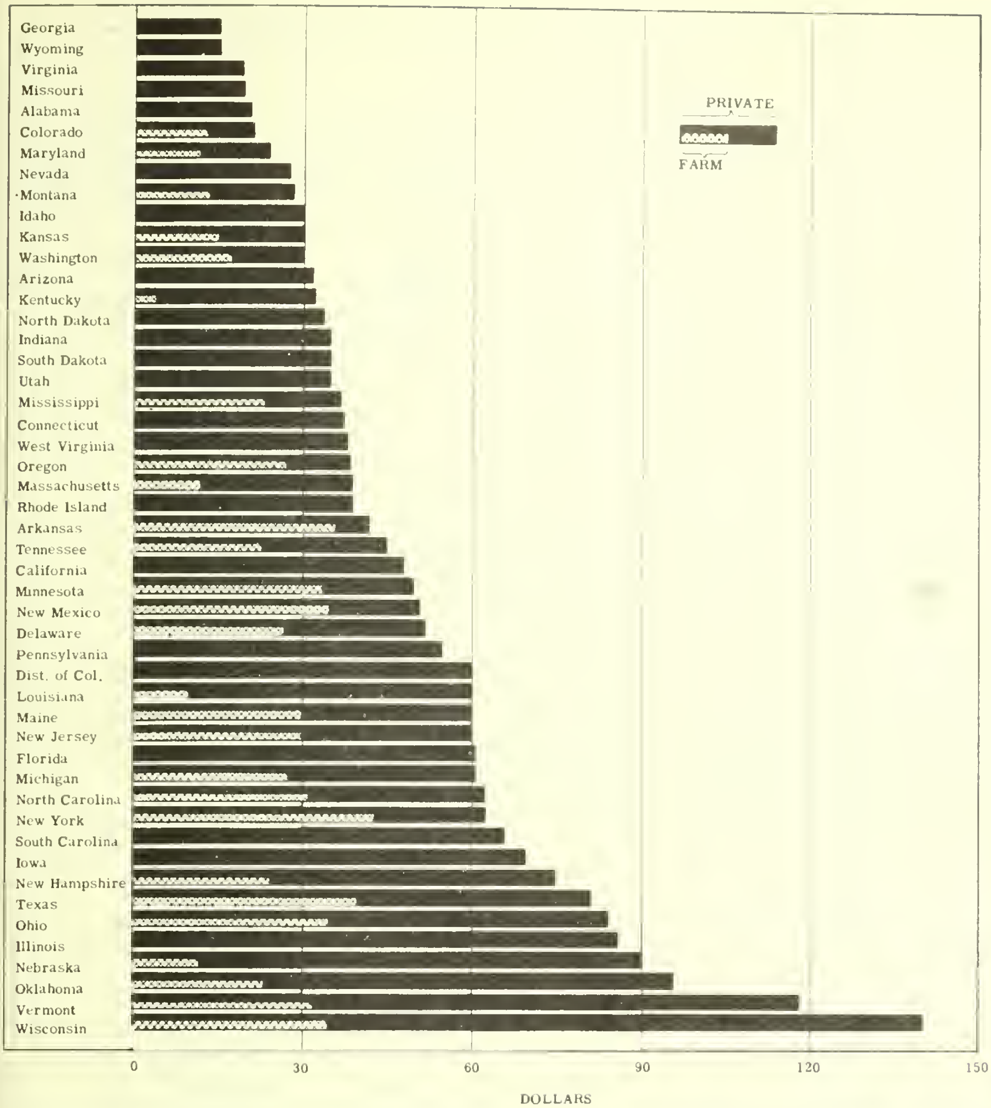


Figure 10.—Comparison of private operation and farm service registration fees on a 12,500-pound stake truck (No. 4).

burdens on certain typical vehicles rather than to include all taxes on all vehicles. Undoubtedly many of the larger trucks traveling in interstate commerce are subject to greater taxes than are reported in this study, and are also subject to additional restrictions and regulations. The larger units apparently have certain offsetting economic advantages, however, or they would not be in use.

Mileage Taxes

Classification and treatment of mileage taxes is something of a problem. Historically, road-user taxes have been subdivided by most authorities into three major groups. The first and most important group consists of fuel taxes and miscellaneous fees incidental to fuel taxation. The second major group, usually termed motor-vehicle revenues, consists of motor-vehicle registration fees and other fees, some of which are not paid annually, such as title fees, drivers' licenses, and other revenues of relatively minor importance. The third group of road-user revenues, motor-carrier taxes, consists of special taxes on for-hire carriers. This last group of taxes has in recent years come to be known as "third structure," a term that is either disliked or rejected by some authorities on highway finance, but it is being used.

These groupings have, in the past, afforded adequate classification of road-user revenues, although there are borderline instances in which there is considerable question as to proper classification. Most common among these is the annual registration fee in States that levy higher plate fees on for-hire carriers. Should the difference between the registration fee in private and for-hire operation be classed as a carrier tax? It has not previously been so classed, under the theory that carriers' vehicles, as a rule, can be expected to operate more miles than vehicles in private operation, and that the higher registration fee for carriers' vehicles serves as an adjustment for the greater mileage.

Another instance of borderline classification is the special fee charged diesel-powered vehicles. An argument can be made for classification of these additional registration fees as fuel taxes. But the important matter is the fairness of the State's tax system, and this is judged primarily on the total taxes levied on any vehicle, provided, of course, that the methods of reporting or collecting the tax do not in themselves impose additional burdens of record keeping or other requirements not reflected in the actual amounts of taxes paid.

The previous article published in 1953 suggested that we may be witnessing the development of a field of highway taxation—mileage and ton-mile taxes—that should not be placed in any of the three historical groups. At that time Idaho, New York, and Oregon had mileage taxes that could not be classed as fuel taxes, as registration fees, or as carrier taxes. Since that time the Idaho tax has been substantially modified, but Colorado and Ohio have joined New York and Oregon in the group of States that have mileage taxes that are avowed attempts to obtain revenues based almost entirely on the vehicles and their use and that make little distinction between operation for profit as a transportation facility and operation incidental to the conduct of any other enterprise. Highway authorities and students of highway and general public finance have shown much interest in these mileage taxes. It is not an exaggeration to say that mileage taxes are at present the subject of more interest and perhaps controversy than any other phase of motor-vehicle taxation.

Mileage taxes may prove to have two great advantages: They should yield considerable amounts of revenue and they should, in theory at least, be imposed at rates that take into account the value of service received and the costs of providing the service. Mileage taxes also have at least two major disadvantages. The most important of these, from the States' point of view, is the cost of administration. The cost of establishing and maintaining complete records, weight stations, and an enforcement staff is substantial and consumes a relatively greater proportion of the revenue collected than do the other two basic revenue producers—the gasoline tax and registration fees. The maintenance of records for filing of reports required under a mileage tax certainly constitutes a substantial burden on the vehicle owner over and above the actual amount of tax paid.

Administration and Application

While this study is based primarily on legislation, it is appropriate to call attention to administration and enforcement as they affect vehicle taxation. The interpretation and application of road-user taxes in one State as compared with another having similar tax laws may easily result in substantial tax differences on similar vehicles in similar service. Rigid insistence that vehicles be registered for the full weights they actually carry (not to be confused with size and weight limit enforcement) would undoubtedly increase the registration revenues of some States. Practices are understood to vary considerably in this respect. In comparing road-user taxes in different States, it is pertinent to examine practices and interpretations in the application of existing statutes. In preparing this article no allowance was made for such differences.

The payment of motor-fuel taxes is another item that merits close examination. Practically all persons familiar with highway finance agree that some of the gasoline used on the

Table 7.—Road-user and personal-property taxes on a single-unit van truck, 19,000 pounds gross vehicle weight (No. 5)

State	Private operation								Contract carrier								
							Rank of State									Rank of State	
	Registration fee	Property tax	Other taxes and fees	Mileage or ton-mile tax	Gasoline tax	Total	Total fees and taxes	Total, excl. prop. tax	Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Mileage or ton-mile tax	Gasoline tax	Total	Total fees and taxes	Total, excl. prop. tax
New England:																	
Maine	$125.00	$65.79	----	----	$161.49	$352.28	9	12	$125.00	$65.79	----	$30.00	----	$269.22	$490.01	15	18
New Hampshire	114.00	----	$47.37	----	115.35	276.72	24	15	114.00	----	$47.37	2.00	----	192.30	355.67	38	30
Vermont	199.50	----	----	----	126.88	326.38	14	6	199.50	----	----	----	----	211.53	411.03	24	20
Massachusetts	57.00	118.00	----	----	115.35	290.35	20	44	57.00	118.00	----	15.00	----	192.30	382.30	31	44
Rhode Island	54.00	83.39	----	----	92.28	229.67	41	48	54.00	83.39	----	5.00	----	153.84	296.23	45	49
Connecticut	57.00	145.92	----	----	138.42	341.34	12	39	57.00	145.92	----	5.00	----	230.76	438.68	21	40
Middle Atlantic:																	
New York	95.00	----	----	$67.50	92.28	254.78	29	22	95.00	----	----	----	$112.50	153.84	361.34	35	28
New Jersey	90.00	----	1.00	----	92.28	183.28	49	41	90.00	----	1.00	----	----	153.84	244.84	49	46
Pennsylvania	90.00	----	----	----	138.42	228.42	42	26	90.00	----	----	----	----	230.76	320.76	42	35
Delaware	81.90	----	----	----	115.35	197.25	46	37	81.90	----	----	----	----	192.30	274.20	47	43
Maryland	75.00	20.00	----	----	138.42	233.42	40	32	75.00	20.00	----	----	----	230.76	325.76	40	38
District of Columbia	74.00	----	1.00	----	138.42	213.42	45	33	74.00	----	1.00	----	----	230.76	305.76	44	39
West Virginia	84.50	18.91	----	----	138.42	241.83	37	29	84.50	18.91	----	23.50	----	230.76	357.67	36	32
Southeastern:																	
Virginia	45.60	69.47	----	----	138.42	253.49	31	40	45.60	69.47	----	240.00	----	230.76	585.83	7	8
North Carolina	152.00	52.03	----	----	161.49	365.52	6	8	266.00	52.03	----	----	----	269.22	587.25	6	6
South Carolina	126.00	94.95	----	----	161.49	382.44	4	11	126.00	94.95	----	150.00	----	269.22	640.17	2	5
Georgia	55.00	66.78	----	----	149.96	271.74	26	36	110.00	66.78	----	25.00	----	249.99	451.77	19	26
Florida	84.70	----	.25	----	161.49	246.44	33	24	84.70	----	.25	.50	250.00	269.22	604.67	4	2
Kentucky	112.00	103.48	.75	----	161.49	377.72	5	18	45.00	103.48	.50	126.00	----	269.22	544.20	10	15
Tennessee	240.00	----	----	----	161.49	401.49	2	1	315.00	----	----	12.50	----	269.22	596.72	5	3
Alabama	50.00	25.46	.50	----	161.49	237.45	39	34	50.00	25.46	.50	----	125.00	269.22	470.18	17	14
Mississippi	94.00	90.12	----	----	161.49	345.61	10	21	187.00	90.12	----	13.00	----	269.22	559.34	8	12
East Central:																	
Ohio	143.85	----	----	----	115.35	259.20	28	20	143.85	----	----	20.00	----	192.30	356.15	37	29
Indiana	50.00	153.11	----	----	92.28	295.39	18	49	50.00	153.11	----	24.00	----	153.84	380.95	32	47
Illinois	159.00	44.71	----	----	115.35	319.06	16	17	159.00	44.71	----	----	----	192.30	396.01	26	31
Michigan	107.80	----	----	----	138.42	246.22	34	25	107.80	----	----	----	50.00	230.76	388.56	28	24
Wisconsin	215.00	----	----	----	138.42	353.42	8	3	215.00	----	----	10.00	----	230.76	455.76	18	13
Minnesota	80.00	----	.25	----	115.35	195.60	47	38	80.00	----	.25	7.50	----	192.30	280.05	46	41
Iowa	190.00	----	----	----	138.42	328.42	13	5	190.00	----	----	5.00	----	230.76	425.76	23	17
Missouri	80.00	37.36	----	----	69.21	186.57	48	47	80.00	37.36	----	25.00	----	115.38	257.74	48	48
Southwestern:																	
Arkansas	67.00	32.10	----	----	149.96	249.06	32	30	67.00	32.10	----	----	----	249.99	349.09	39	37
Louisiana	120.00	----	----	----	161.49	281.49	22	14	240.00	----	----	10.00	----	269.22	519.22	12	7
Oklahoma	145.50	----	----	----	149.96	295.46	17	10	145.50	----	----	2.25	----	249.99	397.74	25	22
Texas	133.00	46.78	----	----	115.35	295.13	19	23	133.00	46.78	----	11.00	----	192.30	383.08	30	33
West Central:																	
North Dakota	46.25	----	40.00	----	138.42	224.67	44	28	46.25	----	40.00	55.00	----	230.76	372.01	33	27
South Dakota	90.00	----	185.00	----	115.35	390.35	3	2	90.00	----	185.00	10.00	----	192.30	477.30	16	11
Nebraska	165.00	39.17	----	----	138.42	342.59	11	9	165.00	39.17	----	15.00	----	230.76	449.93	20	21
Kansas	50.00	89.05	----	----	115.35	254.40	30	46	75.00	89.05	----	10.00	----	192.30	366.35	34	42
Mountain:																	
Montana	50.00	144.92	----	----	161.49	356.41	7	35	50.00	144.92	----	70.00	----	269.92	534.84	11	23
Idaho	55.00	----	----	89.25	138.42	282.67	21	13	55.00	----	----	----	148.75	230.76	434.51	22	16
Wyoming	40.00	63.16	2.50	57.56	115.35	278.57	23	31	40.00	63.16	----	2.50	95.94	192.30	393.90	27	34
Colorado	43.44	63.16	1.00	----	138.42	246.02	35	42	17.50	63.16	1.00	----	232.75	230.76	545.17	9	10
New Mexico	88.80	----	----	----	138.42	227.22	43	27	88.80	----	----	----	----	230.76	319.56	43	36
Arizona	54.05	71.20	----	----	115.35	240.60	38	45	54.05	71.20	----	300.00	----	192.30	617.55	3	4
Utah	65.00	64.89	----	----	115.35	245.24	36	43	65.00	64.89	----	----	----	192.30	322.19	41	45
Nevada	38.50	75.40	151.60	----	138.42	403.92	1	4	38.50	75.40	----	151.60	----	230.76	496.26	14	19
Pacific:																	
Washington	75.00	----	42.30	----	149.96	267.26	27	19	75.00	----	42.30	18.00	----	249.99	385.29	29	25
Oregon	53.90	----	----	127.50	138.42	319.82	15	7	53.90	----	----	----	212.50	230.76	497.16	13	9
California	75.00	----	63.00	----	138.42	276.42	25	16	75.00	----	63.00	367.67	----	230.76	736.43	1	1

highways escapes taxation (by means of refunded taxes), and that the amounts vary greatly between States. There is no way of knowing how much revenue escapes through excessive refund claims, but it is probable that the total is considerable.

For example, the stipulation in this study for the lightweight automobile is 9,500 miles of annual travel at 16.5 miles per gallon, with total gasoline consumption of 576 gallons for the year. Yet the average for *all* vehicles in one State was less than 576 gallons in both 1953 and 1954. This undoubtedly had much to do with the State legislature's conclusion that taxation of ownership is a more certain source of revenue than taxation of use, as evidenced by the increase in fees for registration of lightweight and mediumweight cars to $25 and $35, respectively, and elimination of reduced fees for farm trucks.

Another subject for scrutiny is the collection of diesel-fuel taxes. There are numerous ways of avoiding or evading the tax on diesel fuel, and State authorities charged with collecting it are in general agreement that a considerable amount escapes taxation. The authorities feel that the situation is improving, but the fact that kerosene and some heating oils can be used as diesel fuel makes it seem probable that collection of the tax will remain an administrative problem for some time.

Summary of Comparisons

The foregoing discussion has been of general subjects rather than specific vehicles, except for the remarks on farm rates. It is impractical to include in this article a full discussion of each of the tables and charts presented, but there are comparisons and items of special interest in connection with each of the vehicles that do merit being called to attention.

Vehicle No. 1, lightweight passenger car (table 4; fig. 11).—Registration fees for the lightweight passenger car range from $3 to $30.47, and total road-user taxes from $28.28 to $67.91. Since this vehicle is assumed to use 576 gallons of gasoline a year, each cent of a State's gasoline-tax rate amounts to a levy of $5.76 during the course of a year, or 11 cents a week. The lowest gasoline-tax rate costs the owner of the lightweight passenger car 33 cents a week, the highest about 78 cents a week.

There are 11 States with property taxes alone that exceed the total of the road-user taxes paid in the State ranking lowest. (The property tax shown is for the State capital, however, and undoubtedly exceeds the average for the State in most cases.)

There are only 14 States in which the total road-user taxes on this lightweight automobile exceed $52 per year, or $1 per week. In the State with the highest road-user taxes on this vehicle, the annual amount is $67.91, a little less than 19 cents per day. In the State with the lowest road-user tax rate on the lightweight automobile ($28.28), these taxes amount to a little less than 8 cents per day.

Vehicle No. 2, mediumweight passenger car (table 4).—The mediumweight and lightweight passenger cars are representative of far more than half of all vehicles on the highways. Taxation of automobiles receives less attention than taxation of heavier commercial units, but automobiles, because of their vast number, contribute substantially more in road-user taxes.

Differences between the road-user taxes on lightweight automobiles and on those of medium weight are not great. The registra-

LIGHTWEIGHT PASSENGER CAR (No. 1)

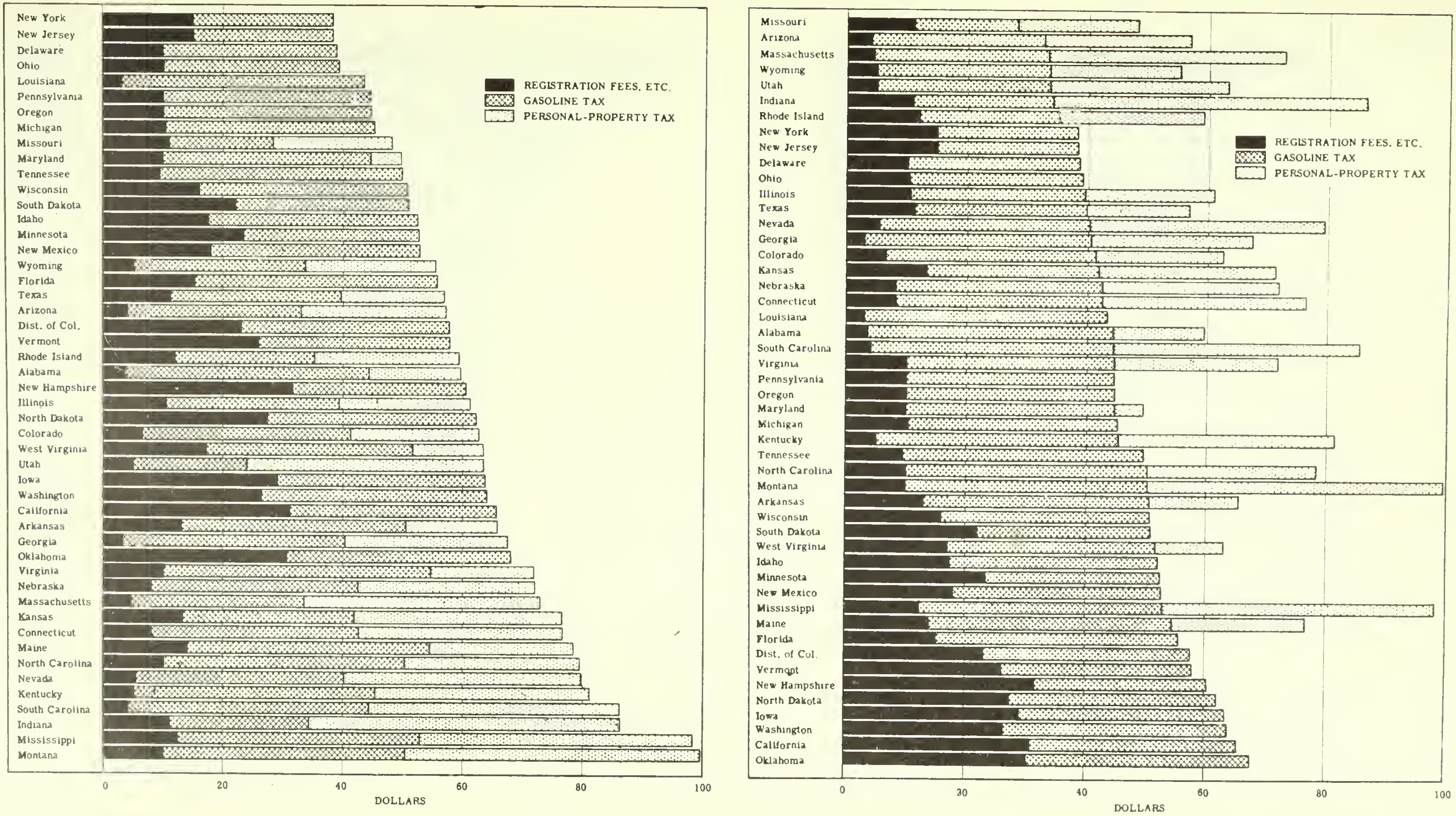


Figure 11.—State road-user and personal-property taxes on a lightweight passenger car (No. 1), ranked according to total taxes (left) and road-user taxes (right).

PICKUP TRUCK (No. 3)

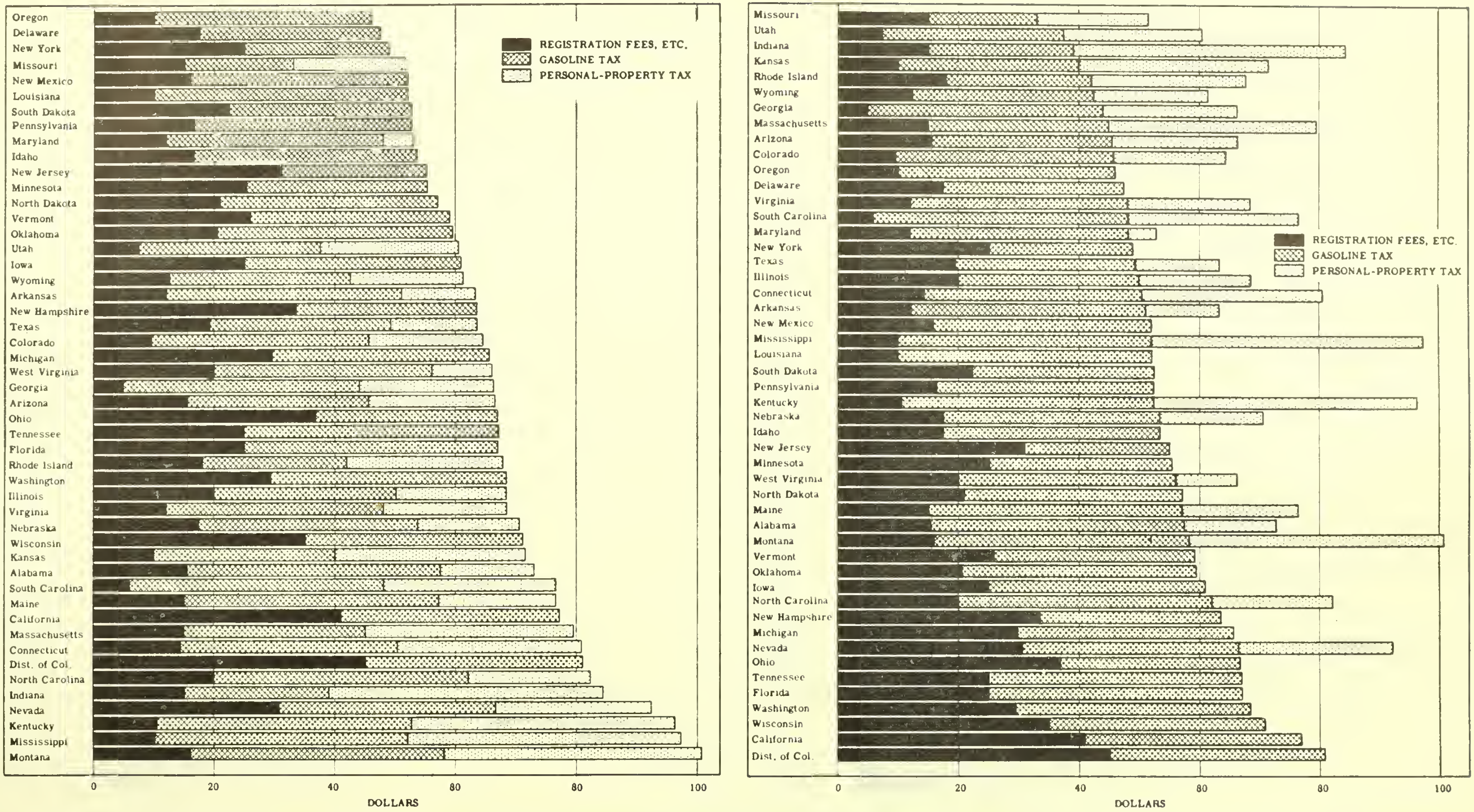


Figure 12.—State road-user and personal-property taxes on a pickup truck (No. 3) in private use, ranked according to total taxes (left) *and road-user taxes* (right).

12,500-POUND STAKE TRUCK (No. 4)

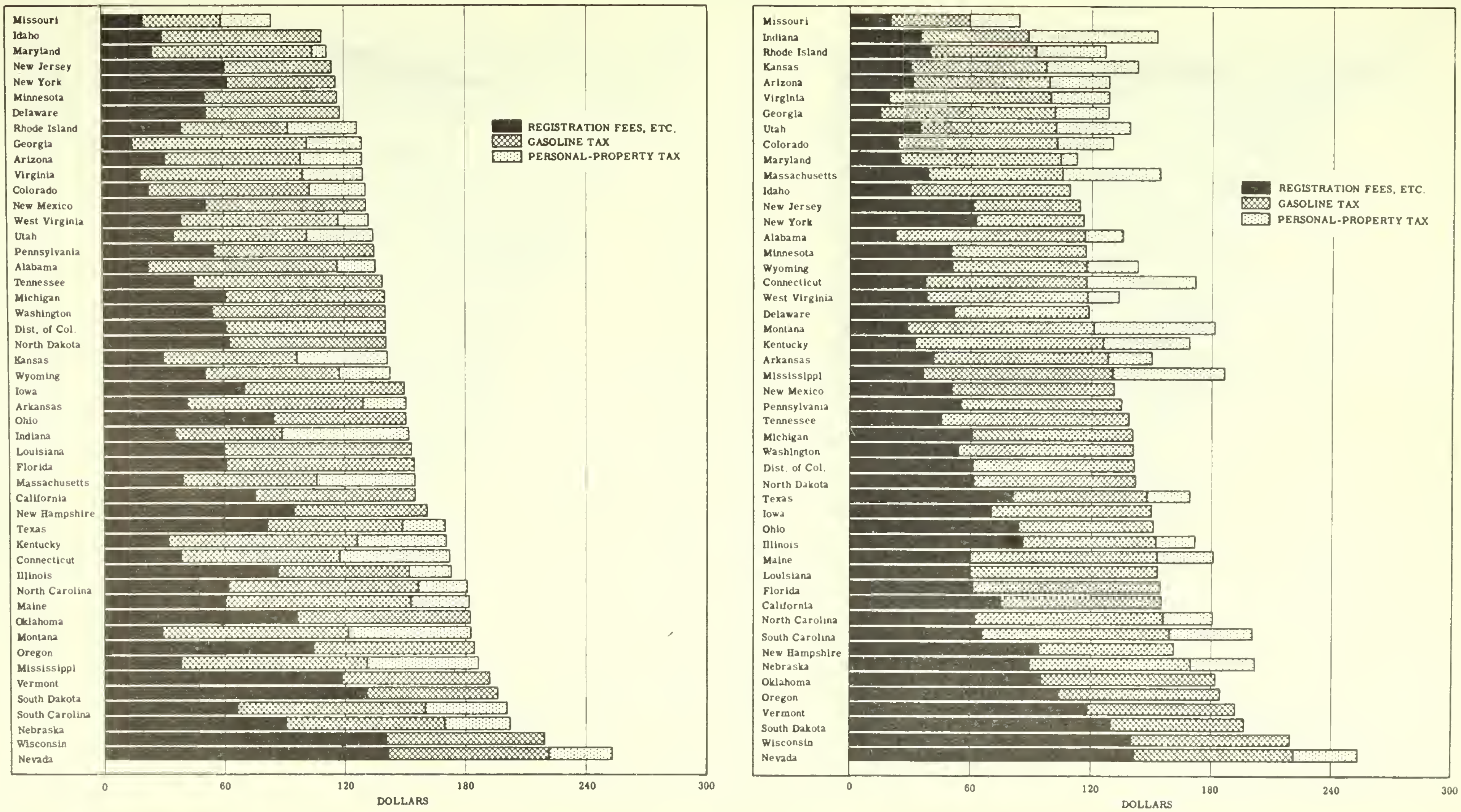


Figure 13.—State road-user and personal-property taxes on a 12,500-pound stake truck (No. 4) in private use, ranked according to total taxes (left) and road-user taxes (right)

19,000-POUND VAN TRUCK (No. 5)

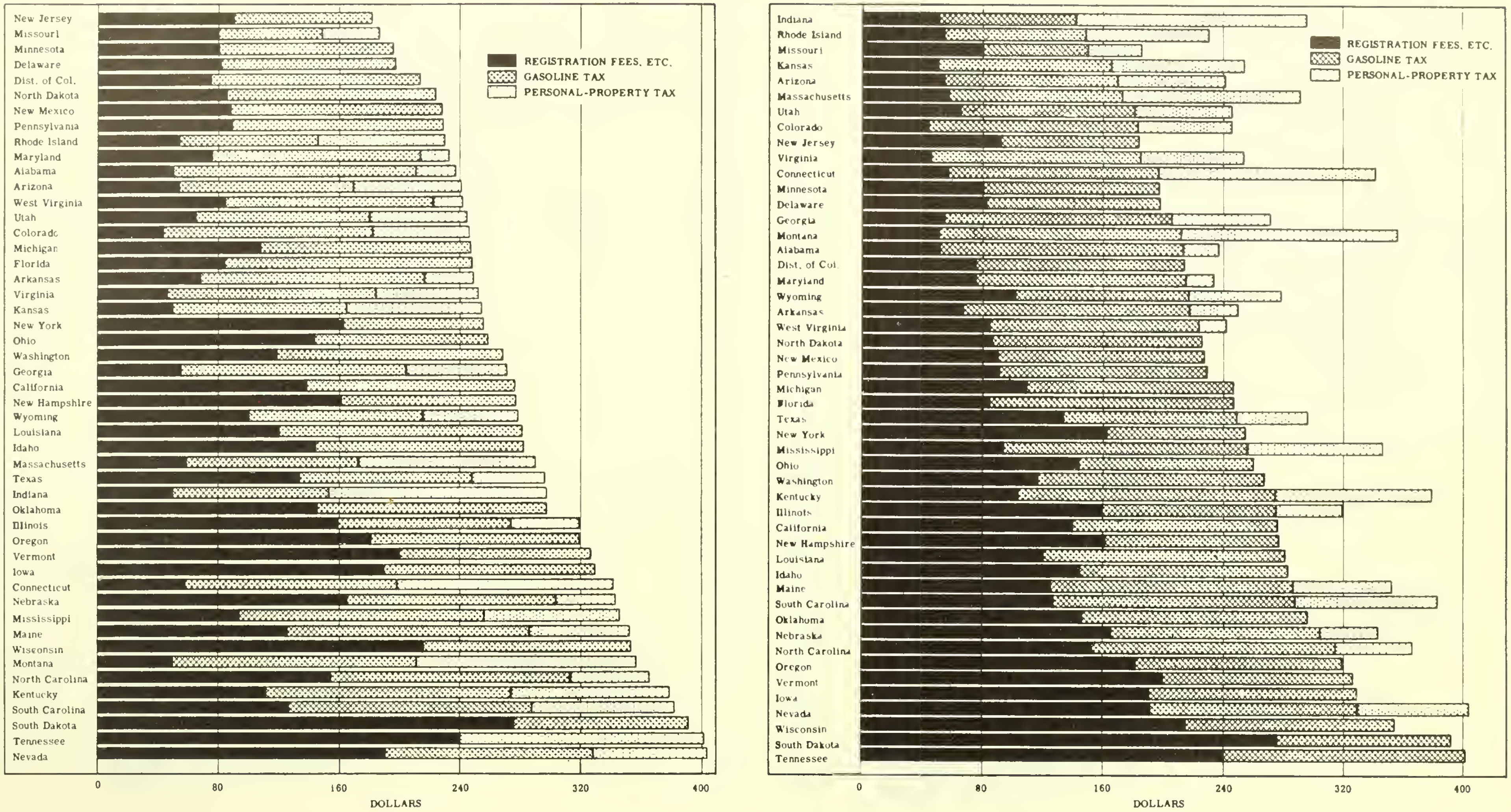


Figure 14.—State road-user and personal-property taxes on a 19,000-pound van truck (No. 5) in private use, ranked according to total taxes (left) and road-user taxes (right).

Table 8.—Road-user and personal-property taxes on a single-unit, three-axle dump truck, 40,000 pounds gross vehicle weight (No. 6)

State	Private operation							
	Registration fee	Property tax	Other taxes and fees	Mileage or ton-mile tax	Gasoline tax	Total	Rank of State	
							Total fees and taxes	Total, excluding property tax
New England:								
Maine	$300.00	$133.39	------	------	$525.00	$958.39	9	19
New Hampshire	240.00	------	$96.04	------	375.00	711.04	38	29
Vermont	420.00	------	------	------	412.50	832.50	23	18
Massachusetts	120.00	235.00	------	------	375.00	730.00	33	46
Rhode Island	127.00	167.40	------	------	300.00	594.40	45	47
Connecticut	200.00	218.88	------	------	450.00	868.88	19	36
Middle Atlantic:								
New York	200.00	------	------	$187.50	300.00	687.50	40	32
New Jersey	240.00	------	1.00	------	300.00	541.00	47	43
Pennsylvania	300.00	------	------	------	450.00	750.00	29	25
Delaware	178.50	------	------	------	375.00	553.50	46	42
Maryland	360.00	50.00	------	------	450.00	860.00	20	20
District of Columbia	202.00	------	1.00	------	450.00	653.00	43	35
West Virginia	242.00	43.64	------	------	450.00	735.64	31	30
Southeastern:								
Virginia	180.00	82.17	------	------	450.00	712.17	37	37
North Carolina	320.00	112.19	------	------	525.00	957.19	11	16
South Carolina	401.00	224.71	------	------	525.00	1,150.71	3	7
Georgia	110.00	135.34	------	------	487.50	732.84	32	39
Florida	187.00	------	.25	------	525.00	712.25	36	28
Kentucky	350.00	214.92	.75	------	525.00	1,090.67	5	13
Tennessee	435.00	------	------	------	525.00	960.00	7	3
Alabama	100.00	25.46	.50	------	525.00	650.96	44	38
Mississippi	271.00	92.15	------	------	525.00	888.15	17	21
East Central:								
Ohio	430.75	------	2.00	150.00	375.00	957.75	10	4
Indiana	200.00	310.45	------	------	300.00	810.45	25	45
Illinois	640.00	86.92	------	------	375.00	1,101.92	4	2
Michigan	382.50	------	------	------	450.00	832.50	22	17
Wisconsin	475.00	------	------	------	450.00	925.00	14	9
Minnesota	350.00	------	.25	------	375.00	725.25	35	27
Iowa	465.00	------	------	------	450.00	915.00	15	10
Missouri	100.00	58.38	------	------	225.00	383.38	48	48
Southwestern:								
Arkansas	200.00	40.18	------	------	487.50	727.68	34	31
Louisiana	240.00	------	------	------	525.00	765.00	28	23
Oklahoma	395.50	------	------	------	487.50	883.00	18	12
Texas	360.00	96.92	------	------	375.00	831.92	24	26
West Central:								
North Dakota	210.00	------	121.50	------	450.00	781.50	27	22
South Dakota	407.50	------	465.00	------	375.00	1,247.50	1	1
Nebraska	405.00	103.51	------	------	450.00	958.51	8	14
Kansas	150.00	128.18	------	------	375.00	653.18	42	44
Mountain:								
Montana	400.00	293.70	------	------	525.00	1,218.70	2	8
Idaho	100.00	------	399.00	------	450.00	949.00	12	5
Wyoming	130.00	128.05	257.50	------	375.00	890.55	16	24
Colorado	211.88	128.05	1.00	------	450.00	790.93	26	33
New Mexico	208.50	------	------	------	450.00	658.50	41	34
Arizona	189.00	144.00	------	------	375.00	708.00	39	41
Utah	200.00	170.15	------	------	375.00	745.15	30	40
Nevada	85.00	149.35	375.30	------	450.00	1,059.65	6	11
Pacific:								
Washington	------	------	------	------	------	--------	------	------
Oregon	119.00	------	360.00	------	450.00	929.00	13	6
California	275.00	------	122.00	------	450.00	847.00	21	15

tion fees for this vehicle range from $3 to $40.19. The $40.19 fee applies in Oklahoma, and this fee combined with the computed $47.06 payment of gasoline tax (the Oklahoma gasoline-tax rate is 6½ cents per gallon) amounts to a total payment of $87.25, the largest total of road-user taxes on this vehicle in any State. However, there is no property tax on motor vehicles in Oklahoma and if property taxes are included for States in which they are imposed, Oklahoma ranks seventeenth in total fees and taxes. Massachusetts, on the other hand, imposes a registration fee of only $9, and with a 5-cent gasoline tax collects a total of $45.20 in road-user taxes on the mediumweight sedan. If the property tax of $56.50 is added, however, the total becomes $101.70—substantially greater than the total of the Oklahoma taxes.

Vehicle No. 3, pickup truck (table 5; fig. 12).—The registration fees for the pickup truck in private use are slightly higher in most States than those on passenger cars. However, large numbers of pickup trucks are registered at reduced farm rates. The lowest registration fee for the pickup truck in any State is $3 (Louisiana), and this rate applies to farm service only. In six States the registration fee for the pickup in ordinary (non-farm) service is less than $10. The highest registration fee for the pickup truck in private (non-farm) service is found in the District of Columbia, $44.

It is probable that pickup trucks outnumber the total of all other trucks in many States. Although the total taxes on each pickup are relatively small compared with taxes on heavier vehicles, the large number of pickups makes this an extremely important category from a revenue standpoint.

Vehicle No. 4, stake truck, 12,500 pounds gross vehicle weight (table 6; fig. 13).—Vehicles in the 12,500-pound group are subject to registration fees which in some States break away rather sharply from the fees charged on automobiles and pickup trucks, although this is not true in all cases. The stake truck can be registered in Georgia and Wyoming for $15, in Virginia for $19.50, and in Missouri for $20. At the other end of the registration-fee scale for this vehicle are Wisconsin and Vermont where the fees for the 12,500-pound stake truck are $140.00 and $118.75, respectively. Here again the farm vehicle rates offer substantial fee reductions in many States, including Vermont and Wisconsin.

In contrast to the heavier trucks, there are large numbers of vehicles registered in the approximate weight range represented by this 12,500-pound unit (the group commonly classed as "1½-ton").

Vehicle No. 5, van truck, 19,000 pounds gross vehicle weight (table 7; fig. 14).—The average registration fee on the 19,000-pound van truck in private service is $112.56. The lowest fee ($44.44 including a $1.00 inspection fee charge) is found in Colorado, and the highest is that of Tennessee, $240. Other road-user taxes—those not included in the motor fuel or registration groups—become more in evidence on reaching the vehicles in the 19,000-pound weight group. In 10 States such additional fees amount to $40 or more on this vehicle in private operation.

Vehicle No. 6, 40,000-pound dump truck (table 8; fig. 15).—The 40,000-pound dump truck, with perhaps minor adjustments of axle spacing, would be permitted to operate in all States except Washington. (Its axle spacing and loading are representative of concrete transit-mix and other special equipment.) The Nevada registration fee on this truck is $85, and there are three States in which the registration fee is $100. The highest fee, $640, is found in Illinois, and the average for all States is $266.94. The registration fees, however, while still important are of relatively reduced significance on vehicles of this gross weight. The mileage and other taxes must be combined with registration fees in order to get a significant comparison.

Vehicle No. 7, 40,000-pound combination (table 10; fig. 16).—The 40,000-pound combination is typical of the largest group of combinations, and is found in substantial numbers even where much heavier vehicles are permitted. It is legal in all States. The average road-user tax payment on it in private operation is $794.74. The Colorado registration fee on this combination is nominal, $22.50; but this low registration fee is accompanied by a substantial mileage tax. In Alabama and Georgia, the only two States that still retain the "manufacturer's rated capacity" basis of assessing registration fees, the registration fees are $75 and $135, respectively, and neither of the two States imposes a mileage tax. However, the gasoline-tax rate in Alabama is 7 cents per gallon and in Georgia, 6½ cents per gallon.

It is interesting to note in table 9 (p. 57) that some States impose on the tractor by far the larger part of the total taxes on the combination, whereas in others the semitrailers carry a relatively large part of the total tax. Although the differences between the taxes on vehicle Number 7 in private operation and the same vehicle in contract service are not discussed in detail here, the variations between the types and the total amounts of taxes on the different services are very great in some States.

(Text continued on page 57)

STATE LEGAL MAXIMUM LIMITS OF MOTOR VEHICLE SIZES AND WEIG

Prepared by the Bureau of Public Roads Jul

ine	State	Width inches[1]	Height ft.-in.	Length—feet[2]				Number of towed units[3]			Axle load—pounds				Gross weight lim	
				Single unit		Truck tractor semi-trailer	Other combination	Semi-trailer	Full trailer	Semi-trailer and full trailer	Single		Tandem		Type of restriction	Formula or equation
				Truck	Bus						Statutory limit	Including statutory enforcement tolerance	Statutory limit	Including statutory enforcement tolerance		
1	Alabama	96	[6]12-6	35	40	50	NP	1	NP	NP	18,000	19,800	36,000	39,600	Table	
2	Arizona	96	13-6	40	40	65	65	1	1	2	18,000		32,000		Table	
3	Arkansas	96	13-6	35	40	50	50	1	1	NP	18,000	[7]18,500	32,000	32,500	Spec. maximum[8]	
4	California	96	13-6	35	[9]35	[10]60	60	NR	NR	NR	18,000		32,000		Table	
5	Colorado	[11]96	12-6	35	40	60	60	1	2	2	18,000		36,000		Formula-spec. limits	800 (L+
6	Connecticut	102	12-6	45	45	45	NP	1	NP	NP	22,400	22,848	36,000	36,720	Spec. lim.-tire cap.	
7	Delaware	96	[6]12-6	35	42	50	60	1	1	2	20,000		36,000		Table-spec. limits[53]	
8	District of Columbia	96	12-6	35	35	50	50	1	1	NP	22,000		38,000		Table	
9	Florida	96	[6]12-6	[12]35	40	50	50	1	1	NP	20,000	22,000	40,000	44,000	Table	
10	Georgia	96	13-6	[13]+39	[13]+45	48	48	1	1	NP	18,000	20,340	36,000	40,680	Spec. maximum[14]	
11	Hawaii	96	13-0	40	40	55	65	1	1	2	24,000		30,800		Formula[55]	800 (L+
12	Idaho	[15]96	14-0	35	[16]40	60	65	1	1	2	[17]18,000		[17]32,000		Table[17]	
13	Illinois	96	13-6	42	42	50	50	1	1	2	[18]18,000		32,000		Spec. lim.-tire cap.	
14	Indiana	96	13-6	36	40	50	50	1	1	2	[19]18,000	[19]19,000	[19]32,000	[19]33,000	Spec. lim.-tire cap.[19]	
15	Iowa	96	[6]12-6	35	[16]40	[20]45	NP	1	NP	NP	18,000	18,540	32,000	32,960	Table	
16	Kansas	96	12-6	35	[16]40	50	50	1	1	NP	18,000		32,000		Table	
17	Kentucky	96	[21]12-6	[22]35	[22]35	[56]48	NP	1	NP	NP	18,000	[57]18,900	32,000	[57]33,600	Spec. lim.-tire cap.[58]	
18	Louisiana	96	[6]12-6	35	[16]40	50	60	1	1	NP	18,000		32,000		Axle lim.-tire cap.	
19	Maine	96	[23]12-6	[24]45	[24]45	[24]45	[24]45	1	1	NP	22,000		32,000		Table-tire cap.[25]	
20	Maryland	96	[6]12-6	55	55	55	55	NR	NR	NR	22,400		[26]40,000		Formula	850 (L+
21	Massachusetts	96	NR	35	[16]40	45	NP	1	NP	NP	22,400		36,000		Table-spec. limits	1,000 (L+
22	Michigan	96	[6]12-6	35	40	55	55	1	1	2	[28]18,000		[29]32,000		Axle lim.-tire cap.	
23	Minnesota	96	[6]12-6	40	40	45	45	1	1	NP	18,000		28,000		Table	
24	Mississippi	96	[6]12-6	35	40	[59]45	45	1	1	NP	18,000		28,650	[30]32,000	Table-tire cap.	
25	Missouri	96	12-6	35	40	45	45	1	1	2	18,000		32,000		Table	
26	Montana	[15]96	13-6	35	40	60	60	1	1	[31]2	18,000		32,000		Table	
27	Nebraska	96	12-6	35	[16]40	50	50	1	1	NP	18,000	18,900	32,000	33,600	Table	
28	Nevada	96	NR	NR	NR	NR	NR	NR	NR	NR	18,000	18,900	32,000	33,600	Table	
29	New Hampshire	96	13-6	35	[16]40	45	45	NR	NR	NR	22,400		36,000		Tables-spec. limits	
30	New Jersey	96	13-6	35	[33]35	45	[34]50	1	1	NP	22,400	23,520	32,000	33,600	Spec. limits	
31	New Mexico	[35]96	13-6	40	40	65	65	1	1	2	21,600		34,320		Table	
32	New York	96	13-0	35	[36]35	50	50	1	1	NP	22,400		36,000		Formula	34,000+
33	North Carolina	96	[6]12-6	35	[16]40	48	48	1	1	NP	18,000	19,000	36,000	38,000	Spec. limits	
34	North Dakota	96	12-6	[12]35	[16]40	50	50	1	1	NP	18,000		30,000		Formula	650 & 750 (L+
35	Ohio	96	[6]12-6	35	[16]40	50	60	1	NR	NR	19,000		[37]31,500		Formula	800 (L+4
36	Oklahoma	96	13-6	35	45	50	50	1	1	NP	18,000		32,000		Table	
37	Oregon	96	[38]12-6	35	[30]40	[30]55	[30]60	1	1	[30]2	[39]18,000		[39]32,000		Table[40]	
38	Pennsylvania	96	[6]12-6	35	[16]40	50	50	1	1	NP	22,400	23,072	36,000	37,080	Spec. limits[41]	
39	Puerto Rico[54]															
40	Rhode Island	102	12-6	40	40	50	50	1	1	NP	22,400		NS		Spec. limits	
41	South Carolina	96	12-6	[12]35	[16]40	50	50	1	1	NP	20,000		32,000		Table	
42	South Dakota	96	13-0	35	[16]40	50	50	1	1	NP	18,000		32,000		Table	
43	Tennessee	96	12-6	35	40	45	45	1	1	NP	18,000		32,000		Table	
44	Texas	96	13-6	35	40	50	50	1	1	NP	18,000	18,900	32,000	33,600	Table	
45	Utah	96	14-0	45	45	60	60	NR	NR	NR	18,000		33,000		Table	
46	Vermont	96	12-6	50	50	50	50	1	1	NP	NS		NS		Spec. lim.-tire cap.	
47	Virginia	96	[6]12-6	35	[30]40	50	50	1	1	NP	[18]18,000		[60]32,000		Table	
48	Washington	96	[5]12-6	35	[16]40	60	60	1	1	[47]2	18,000	[48]18,500	32,000	[48]33,000	Table-spec. lim.[49]	
49	West Virginia	96	[6]12-6	35	[16]40	45	45	1	1	NP	18,000	18,900	32,000	33,600	Table	
50	Wisconsin	96	[6]12-6	35	40	50	50	1	1	NP	18,000	[50]19,500	30,000	32,000	Table-formula[51]	1,000 (L
51	Wyoming	96	13-6	40	40	60	60	1	1	2	18,000		32,000	[52]36,000	Table	
	AASHO Policy	96	12-6	35	[16]40	50	60	1	1	NP	18,000		32,000		Table	1.025 (L+24
mber of States	Higher	2	20	15	28	14	5	5	7	20	30		29		Formula 6	
	Same	48	30	35	18	22	10	45	38	30	20		16		Table 31	
	Lower	0	0	0	4	14	35	0	5	0	0		5		Specified limits 13	

NP—Not permitted. NR—Not restricted. NS—Not specified.

[1] Various exceptions for farm and construction equipment; public utility vehicles; urban, suburban, and school buses; haulage of agricultural and forest products; at wheels of vehicles; for safety accessories, and on designated highways.

[2] Various exceptions for utility vehicles and loads.

[3] When not specified, limited to number possible in practical combinations within permitted length limits.

[4] Legally specified or established by administrative regulation.

[5] Computed under the following conditions to permit comparison on a uniform basis between States with different types of regulation:

A. Front axle load of 8,000 pounds.

B. Maximum practical wheelbase within applicable length limits:

(1) Minimum front overhang of 3 feet.

(2) In the case of a 4-axle truck-tractor semitrailer, rear overhang computed as necessary to distribute the maximum possible uniform load on the maximum permitted length of semitrailer to the single drive-axle of the tractor and to the tandem axles of the semitrailer, within the permitted load limits of each.

(3) In the case of a combination having 5 or more axles, minimum possible combined front and rear overhang assumed to be 5 feet, with maximum practical load on maximum permitted length of semitrailer, subject to control of loading on axle groups and on total wheelbase as applicable.

C. Including statutory enforcement tolerances as applicable.

[6] Auto transports 13 feet 6 inches.

[7] Does not apply to combinations of adjacent load-carrying single axles.

[8] 56,000 pounds on load-carrying axles, exclusive of steering-axle load.

[9] On specific routes in urban or suburban service under special permit from P.U.C. 40 feet.

[10] Limited by 40-foot maximum length of semitrailer to 55-foot practical maximum length in combination.

[11] Buses 102 inches

[12] Three-axle vehicles 40 feet.

[13] Truck 39 feet 6 1/2 inches; Bus 45 feet 2.4 inches

[14] 63,280 pounds maximum, except on roads under Rural Roads Authority 56,000 pounds maximum.

[15] Buses 102 inches on highways of surfaced width at least 20 feet or otherwise as administratively authorized.

[16] Less than three axles 35 feet.

[17] Special limits for vehicles hauling timber and timber products, ores, conc
tural products including livestock: single axle 18,900 pounds, tandem axle 37
vehicle with 3 or 4 axles permitted 56,000 pounds maximum at 21-foot axle spac
permitted 79,000 pounds maximum at 43-foot axle spacing.

[18] On designated highways; 16,000 pounds on other highways.

[19] On designated highways, single axle 22,400 pounds, tandem axle 36,000 pour
total of all excesses of weight under one or more limitations of axle load and

[20] Auto transports with load permitted overall length of 47 feet under annual

[21] On designated highways; 11 feet 6 inches on other highways.

[22] On designated highways; trucks 26.5 feet and buses 30 feet on other highwa

[23] Including load 14 feet.

[24] Including load 46 feet 6 inches.

[25] Trucks hauling forest products, brakes on 3 axles at 18-foot minimum spac
construction materials, brakes on 3 axles at 16-foot minimum spacing 48,000 po

[26] Spaced less than 48 inches 36,000 pounds.

[27] Subject to axle and tabular limits.

[28] Single axle spaced less than 9 feet from nearest axle limited to 13,000 p

[29] On designated highways only and limited to one tandem axle in combination;

[30] On designated highways only.

[31] Semitrailer and semitrailer converted to full trailer by means of a dolly.

[32] Dual-drive axles; otherwise 40,000 pounds.

[33] Or as prescribed by P.U.C.

[34] Exception for poles, pilings, structural units, etc., permitted 70 feet.

[35] On designated highways 102 inches.

[36] Trackless trolleys and 3-axle buses 40 feet.

[37] Spaced less than 4 feet 24,000 pounds.

[38] Certain types of vehicles and commodities under special permit on designa

COMPARED WITH AASHO STANDARDS

licable to:		Specified maximum gross weight—pounds[4]						Practical maximum gross weight—pounds[5]						Line
		Truck		Truck-tractor semitrailer			Other combination	Truck		Truck-tractor semitrailer			Other combination	
y up : es	Total wheel base only	2-axle	3-axle	3-axle	4-axle	5-axle		2-axle	3-axle	3-axle	4-axle	5-axle		
	X							27,800	47,600	47,600	60,010	64,650	NP	1
18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,800	2
								26,500	40,500	45,000	59,000	65,000	65,000	3
18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,000	4
	X	30,000	46,000					26,000	44,000	44,000	62,000	75,000	76,000	5
		32,000	50,000	50,000	60,000	60,000	NP	30,848	44,720	51,000	61,200	61,200	NP	6
X		30,000	40,000	48,000	60,000	60,000	60,000	28,000	40,000	48,000	56,350	60,000	60,000	7
X								30,000	46,000	52,000	58,450	61,490	64,650	8
	X							30,000	52,000	52,000	65,200	71,115	71,115	9
							63,280	28,340	48,680	48,680	63,280	63,280	63,280	10
18'	Over 18'							32,000	38,800	56,000	62,800	69,600	78,000	11
X								26,000	40,000	44,000	58,000	72,000	76,800	12
		36,000	41,000	45,000	59,000	68,000	72,000	26,000	40,000	44,000	58,000	58,000	72,000	13
							72,000	27,000	41,000	45,000	59,000	73,000	73,000	14
X								26,540	40,960	45,080	59,500	58,657	NP	15
X								26,000	40,000	44,000	55,470	63,890	63,890	16
		36,000	50,000	54,000	59,640	59,640	NP	26,900	41,600	45,800	59,640	59,640	NP	17
								26,000	40,000	44,000	58,000	72,000	76,000	18
X		32,000	50,000	50,000	50,000	50,000	50,000	30,000	40,000	50,000	50,000	50,000	50,000	19
	X					65,000	65,000	30,400	48,000	52,800	65,000	65,000	65,000	20
	X	[27]46,000	[27]60,000	[27]60,000	[27]60,000	[27]60,000	NP	30,400	44,000	52,800	60,000	60,000	NP	21
X								25,000	[30]40,000	44,000	[30]58,000	[30]66,000	[30]102,000	22
X								26,000	36,000	44,000	54,000	65,500	65,500	23
								26,000	36,650	44,000	[30]54,650	[30]55,980	[30]55,980	24
X								26,000	40,000	44,000	55,470	60,010	60,010	25
18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,000	26
22'	Over 22'							26,780	41,200	45,320	57,134	66,590	66,590	27
18'	Over 18'							26,900	41,500	45,800	60,000	74,000	76,800	28
	X	33,400	[32]47,500	52,800	66,400			30,400	44,000	52,800	66,400	66,400	66,400	29
		30,000	40,000	60,000	60,000	60,000	60,000	31,500	41,600	55,040	63,000	63,000	63,000	30
18'	Over 18'							29,600	42,320	51,200	63,920	76,640	86,400	31
	X							30,400	44,000	52,800	65,000	65,000	65,000	32
		31,500	46,200	46,200	58,800	58,800	58,800	27,000	46,000	46,000	58,800	58,800	58,800	33
18'	Over 18'							26,000	38,000	44,000	56,000	63,750	63,750	34
	X							27,000	39,500	46,000	58,500	71,000	78,000	35
X								26,000	40,000	44,000	55,470	60,000	60,000	36
18'	Over 18'					72,000	76,000	26,000	40,000	44,000	58,000	72,000	76,000	37
		33,000	47,000	50,000	60,000	60,000	62,000	31,072	45,080	51,500	61,800	61,800	63,860	38
														39
		[42]36,000	[43]44,000	[44]50,000	[45]60,000	50,000	88,000	30,400	44,000	50,000	60,000	60,000	88,000	40
X								28,000	40,000	48,000	60,000	66,839	71,115	41
X								26,000	40,000	44,000	55,470	61,490	64,650	42
	X							26,000	40,000	44,000	55,980	55,980	43,500	43
X								26,900	41,600	45,800	57,844	61,340	61,340	44
X								26,000	41,000	44,000	59,000	74,000	79,900	45
		30,000	40,000	50,000	[46]60,000	[46]60,000	[46]60,000	30,000	40,000	50,000	60,000	60,000	60,000	46
X						[30]56,800	[30]56,800	26,000	40,000	44,000	56,800	56,800	56,800	47
18'	Over 18'	28,000	36,000	46,000	60,000	68,000	72,000	26,000	36,000	44,000	60,000	68,000	72,000	48
X								26,900	41,600	45,800	57,844	63,840	63,840	49
X								27,500	40,000	47,000	59,500	68,000	68,000	50
X								26,000	44,000	44,000	62,000	73,000	73,000	51
X								26,000	40,000	44,000	55,470	61,490	71,900	
18	19							29 21 0	25 19 6	29 21 0	43 4 3	34 2 14	18 0 32	

ggregates, and agricul-
gross weight table:
with 5 or more axles

ce of 1,000 pounds on
t.

ounds; trucks hauling

26,000 pounds.

up to 13 feet 6 inches.

[39] Logging vehicles permitted 3-foot wheelbase tolerance, 19,000-pound single axle, 34,000-pound tandem axle.

[40] Governs gross weight permitted on highways designated by resolution of State highway commission.

[41] Single unit truck with 4 axles permitted 60,000 pounds.

[42] Axles spaced less than 6 feet 32,000 pounds; less than 12 feet 36,000 pounds; 12 feet or more gross weight governed by axle limit.

[43] Single vehicle with 3 or more axles spaced less than 16 feet 40,000 pounds; less than 20 feet 44.000 pounds; 20 feet or more governed by axle limit.

[44] Tractor semitrailer with 3 or more axles spaced less than 22 feet 46,000 pounds; not less than 27 feet 50,000 pounds.

[45] Axles spaced 27 feet or more.

[46] Tandem axles on trailer equipped with adequate brakes.

[47] Under State highway commission rules.

[48] Within discretion of enforcement officer.

[49] Vehicles hauling logs permitted wheelbase and gross weight tolerances. Discretionary enforcement tolerances not included in computation of practical maximum gross weights.

[50] Axle load 21,000 pounds on 2-axle trucks hauling unmanufactured forest products.

[51] On Class A highways.

[52] Based on ruling of Attorney General.

[53] Legal limit 60,000 pounds.

[54] Withheld pending approval of new statutes.

[55] 700 (L+40) under 18'-800(L+40) 18 feet and over 900(L+40) on highways having no structure with span of 20 feet or over.

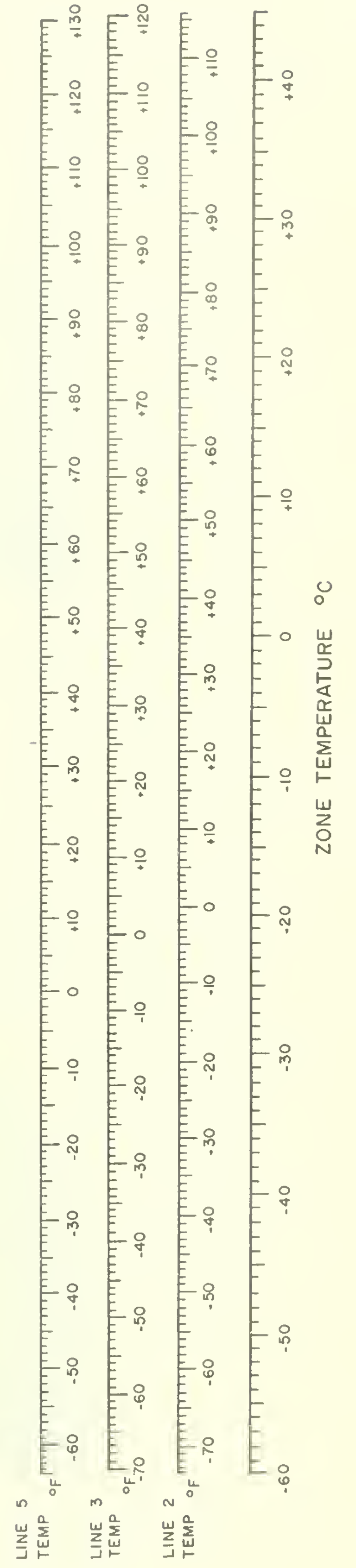
[56] Class AA highways; 45 feet on other highways.

[57] Class AA highways only.

[58] Maximum gross weight on Class A highways 42,000 pounds; on Class B highways 30,000 pounds.

[59] Auto transports permitted 50 feet

[60] Vehicles registered before July 1, 1956 permitted limits in effect January 1, 1956 for life of vehicle.



40,000-POUND TANDEM-AXLE DUMP TRUCK (No. 6)

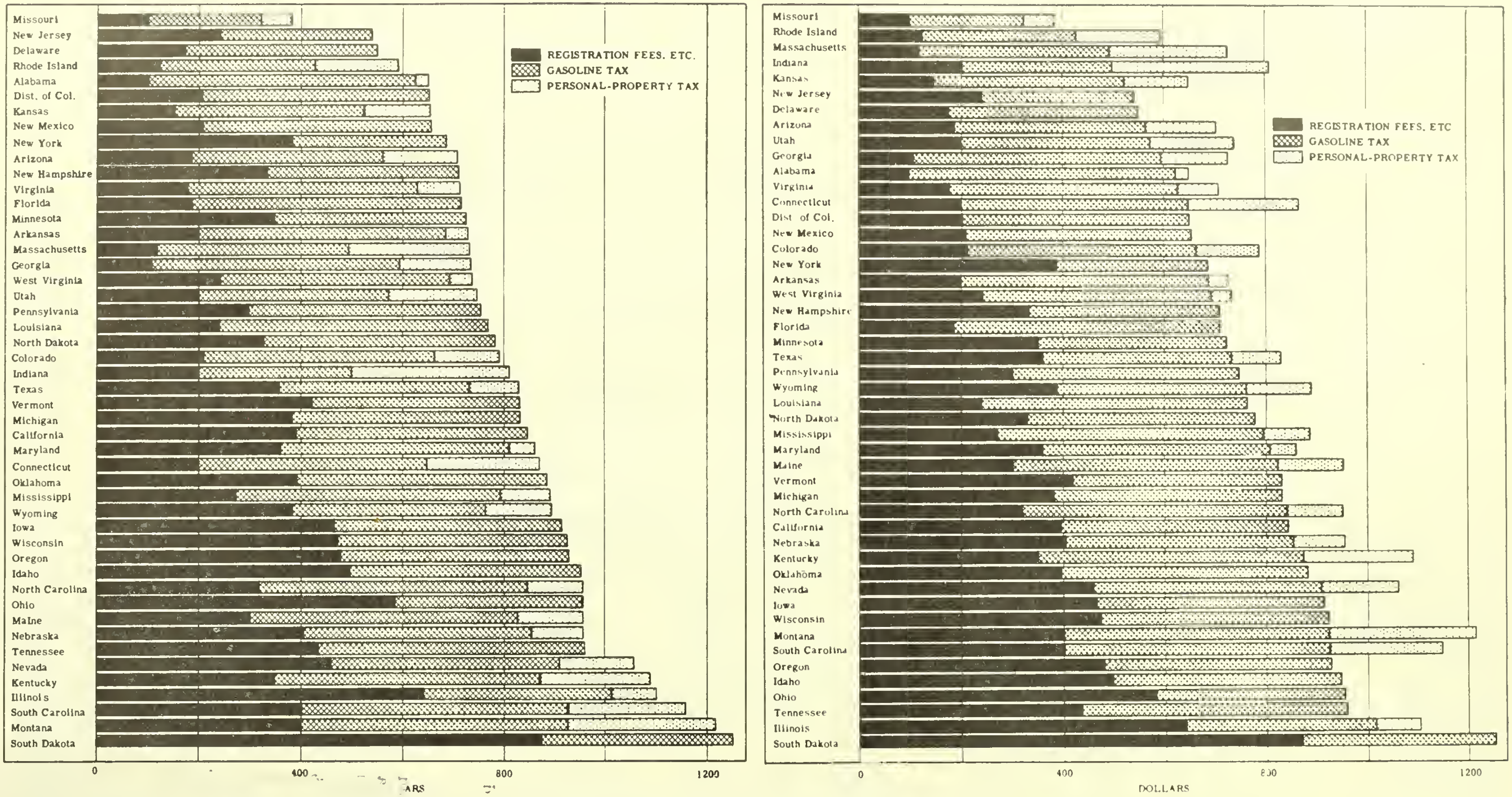


Figure 15.—State road-user and personal-property taxes on a 40,000-pound tandem-axle dump truck (No. 6) in private use, ranked according to total taxes (left) and road-user taxes (right).

Table 9.—Road-user and personal-property taxes on the tractor and the semitrailer of a three-axle combination, 40,000 pounds gross vehicle weight (No. 7), in private operation

State	Tractor truck						Semitrailer				
	Registration fee	Property tax	Other taxes and fees	Milcage or ton-mile tax	Gasoline tax	Total	Registration fee	Property tax	Other taxes and fees	Mileage or ton-mile tax	Total
New England:											
Maine	$300.00	$70.39	--------	--------	$560.00	$930.39	$5.00	$117.00	--------	--------	$122.00
New Hampshire	240.00	--------	$50.68	--------	400.00	690.68	--------	--------	$46.80	--------	46.80
Vermont	420.00	--------	--------	--------	440.00	860.00	15.00	--------	--------	--------	15.00
Massachusetts	120.00	125.00	--------	--------	400.00	645.00	2.00	116.00	--------	--------	118.00
Rhode Island	127.00	87.27	--------	--------	320.00	534.27	2.00	80.60	--------	--------	82.60
Connecticut	200.00	115.52	--------	--------	480.00	795.52	--------	123.88	--------	--------	123.88
Middle Atlantic:											
New York	79.00	--------	--------	$400.00	320.00	799.00	162.50	--------	--------	--------	162.50
New Jersey	110.00	--------	1.00	--------	320.00	431.00	90.00	--------	1.00	--------	91.00
Pennsylvania	190.00	--------	--------	--------	480.00	670.00	125.00	--------	--------	--------	125.00
Delaware	95.70	--------	--------	--------	400.00	495.70	77.30	--------	--------	--------	77.30
Maryland	35.00	30.00	--------	--------	480.00	545.00	100.00	15.00	--------	--------	115.00
District of Columbia	74.00	--------	1.00	--------	480.00	555.00	74.00	--------	1.00	--------	75.00
West Virginia	227.00	18.91	--------	--------	480.00	725.91	15.00	26.18	--------	--------	41.18
Southeastern:											
Virginia	30.00	74.33	--------	--------	480.00	584.33	150.00	68.64	--------	--------	218.64
North Carolina	176.00	57.55	--------	--------	560.00	793.55	144.00	96.60	--------	--------	240.60
South Carolina	101.00	128.71	--------	--------	560.00	789.71	96.00	109.72	--------	--------	205.72
Georgia	25.00	73.41	--------	--------	520.00	618.41	110.00	65.98	--------	--------	175.98
Florida	86.90	--------	.25	--------	560.00	647.15	109.50	--------	.25	--------	109.75
Kentucky	350.00	111.44	.75	--------	560.00	1,022.19	--------	103.48	--------	--------	103.48
Tennessee	435.00	--------	--------	--------	560.00	995.00	--------	--------	--------	--------	--------
Alabama	50.00	135.16	.50	--------	560.00	745.66	25.00	124.80	.50	--------	150.30
Mississippi	271.00	81.41	--------	--------	560.00	912.41	11.00	75.74	--------	--------	86.74
East Central:											
Ohio	152.25	--------	2.00	400.00	400.00	954.25	132.65	--------	--------	--------	132.65
Indiana	215.00	85.20	--------	--------	320.00	620.20	--------	78.63	--------	--------	78.63
Illinois	640.00	41.45	--------	--------	400.00	1,081.45	--------	21.81	--------	--------	21.81
Michigan	154.05	--------	--------	--------	480.00	634.05	142.35	--------	--------	--------	142.35
Wisconsin	475.00	--------	--------	--------	480.00	955.00	10.00	--------	--------	--------	10.00
Minnesota	350.00	--------	.25	--------	400.00	750.25	10.00	--------	.25	--------	10.25
Iowa	435.00	--------	--------	--------	480.00	915.00	60.00	--------	--------	--------	60.00
Missouri	300.00	35.02	--------	--------	240.00	575.02	7.00	37.36	--------	--------	44.36
Southwestern:											
Arkansas	200.00	34.56	--------	--------	520.00	754.56	5.00	29.40	--------	--------	34.40
Louisiana	140.00	--------	--------	--------	560.00	700.00	120.00	--------	--------	--------	120.00
Oklahoma	50.50	--------	--------	--------	520.00	570.50	295.50	--------	--------	--------	295.50
Texas	154.00	47.95	--------	--------	400.00	601.95	117.00	42.05	--------	--------	159.05
West Central:											
North Dakota	420.00	--------	135.00	--------	480.00	1,035.00	--------	--------	--------	--------	--------
South Dakota	102.50	--------	215.00	--------	400.00	717.50	81.00	--------	185.00	--------	266.00
Nebraska	440.00	45.81	--------	--------	480.00	965.81	1.00	78.33	--------	--------	79.33
Kansas	300.00	93.85	10.00	--------	400.00	803.85	20.00	82.20	--------	--------	102.20
Mountain:											
Montana	60.00	154.97	--------	--------	560.00	774.97	40.00	143.13	--------	--------	183.13
Idaho	100.00	--------	--------	532.00	480.00	1,112.00	2.00	--------	--------	--------	2.00
Wyoming	40.00	67.57	2.50	158.30	400.00	668.37	40.00	62.40	2.50	$182.50	287.40
Colorado	17.50	67.57	1.00	--------	480.00	566.07	5.00	62.40	--------	837.44	904.84
New Mexico	103.50	--------	--------	--------	480.00	583.50	73.00	--------	--------	--------	73.00
Arizona	55.35	76.20	--------	--------	400.00	531.55	51.45	70.40	--------	--------	121.85
Utah	200.00	67.77	--------	--------	400.00	667.77	5.00	82.92	--------	--------	87.92
Nevada	39.50	126.75	155.00	--------	480.00	801.25	36.50	117.00	144.80	--------	298.30
Pacific:											
Washington	105.00	--------	39.20	--------	520.00	664.20	55.00	--------	44.00	--------	99.00
Oregon	56.00	--------	--------	760.00	480.00	1,296.00	51.10	--------	--------	--------	51.10
California	75.00	--------	67.00	--------	480.00	622.00	108.00	--------	60.00	--------	168.00

Vehicle No. 8, 50,000-pound gasoline-powered combination (table 11; fig. 17).—The 50,000-pound combination is now legal in all States except Kentucky, and can be operated in Kentucky with reduced load. The average registration fee for this combination is $316.44 when in private service, but in contract service the average registration fee is $342.61. The highest annual total of road-user taxes in private service is $2,772.88, and in contract service the highest total of road-user taxes is $2,793.83.

Figure 21 (p. 66) compares for this combination and for the 40,000-pound combination, the tax costs in cents-per-mile of private and contract operation.

Vehicle No. 9, 50,000-pound diesel-powered combination (table 12; fig. 18).—The 50,000-pound diesel-powered combination, empty, is somewhat heavier than the 50,000-pound gasoline combination and thus carries a correspondingly smaller payload when operating at full capacity. (It is also a somewhat more expensive combination than the one with gasoline power and is subject to greater property taxes in many States.) Offset against this, however, is its substantial saving in total fuel cost, including tax. (The diesel combination is assumed to save one-third in fuel gallonage.) The saving in fuel is highly desirable from almost any standpoint, but the resulting reductions in fuel-tax contributions for the support of highways poses a problem. Seven States have already imposed higher rates on diesel fuel in an effort to obtain a tax contribution similar to that of gasoline-powered vehicles. Vermont does not tax diesel fuel but does impose an equalization fee. The relation between taxes paid on a gasoline-powered combination of 50,000 pounds gross weight and a similar diesel-powered combination may be developed by comparison of data in tables 11 and 12.

A comparison of the tax costs in cents per mile for private use and contract operation is given in figure 22 (p. 67) for this combination (vehicle No. 9), and also for vehicle Nos. 10 and 11, the heaviest diesel-powered combinations.

This study was made in late 1955 and early 1956, and does not take into consideration State legislation of the 1956 sessions. Size and weight limitations reported on page 38 do not agree in every instance with the table on pages 54–55, which shows State legal maximum limits of motor-vehicle sizes and weights as of July 1, 1956.

(*Tables 10–13 and figures 16–22 follow*)

Table 10.—Road-user and personal-property taxes on a three-axle tractor-semitrailer combination, 40,000 pounds gross vehicle weight (No. 7)

State	Private operation								Contract carrier								
							Rank of State									Rank of State	
	Registration fee	Property tax	Other taxes and fees	Mileage or ton-mile tax	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax	Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Mileage or ton-mile tax	Gasoline tax	Total	Total fees and taxes	Total, excluding property tax
New England:																	
Maine	$305.00	$187.39	------	--------	$560.00	$1,052.39	8	17	$305.00	$187.39	------	$30.00	--------	$560.00	$1,082.39	19	24
New Hampshire	240.00	------	$97.48	--------	400.00	737.48	40	29	240.00	------	$97.48	2.00	--------	400.00	739.48	41	35
Vermont	435.00	------	------	--------	440.00	875.00	24	15	435.00	------	------	------	--------	440.00	875.00	31	25
Massachusetts	122.00	241.00	------	--------	400.00	763.00	35	46	122.00	241.00	------	15.00	--------	400.00	778.00	37	47
Rhode Island	129.00	167.87	------	--------	320.00	616.87	47	49	129.00	167.87	------	5.00	--------	320.00	621.87	47	49
Connecticut	200.00	239.40	------	--------	480.00	919.40	21	33	200.00	239.40	------	5.00	--------	480.00	924.40	28	38
Middle Atlantic:																	
New York	241.50	------	------	$400.00	320.00	961.50	18	11	241.50	------	------	------	$400.00	320.00	961.50	25	20
New Jersey	200.00	------	2.00	--------	320.00	522.00	49	47	200.00	------	2.00	------	--------	320.00	522.00	49	48
Pennsylvania	315.00	------	------	--------	480.00	795.00	28	22	315.00	------	------	------	--------	480.00	795.00	35	31
Delaware	173.00	------	------	--------	400.00	573.00	48	43	173.00	------	------	------	--------	400.00	573.00	48	44
Maryland	135.00	45.00	------	--------	480.00	660.00	42	41	135.00	45.00	------	------	--------	480.00	660.00	43	42
District of Columbia	148.00	------	2.00	--------	480.00	630.00	45	40	148.00	------	2.00	------	--------	480.00	630.00	46	41
West Virginia	242.00	45.09	------	--------	480.00	767.09	33	32	242.00	45.09	------	39.50	--------	480.00	806.59	33	34
Southeastern:																	
Virginia	180.00	142.97	------	--------	480.00	802.97	27	36	180.00	142.97	------	480.00	--------	480.00	1,282.97	5	4
North Carolina	320.00	154.15	------	--------	560.00	1,034.15	11	14	560.00	154.15	------	------	--------	560.00	1,274.15	7	7
South Carolina	197.00	238.43	------	--------	560.00	995.43	13	27	197.00	238.43	------	350.00	--------	560.00	1,345.43	4	10
Georgia	135.00	139.39	------	--------	520 00	794.39	29	38	262.50	139.39	------	25.00	--------	520.00	946.89	27	29
Florida	196.40	------	.50	--------	560.00	756.90	38	28	232.90	------	.50	.50	200.00	560.00	993.90	21	16
Kentucky	350.00	214.92	.75	--------	560.00	1,125.67	3	13	150.00	214.92	.50	325.00	--------	560.00	1,250.42	9	15
Tennessee	435.00	------	------	--------	560.00	995.00	14	7	560.00	------	------	17.50	--------	560.00	1,137.50	11	5
Alabama	75.00	259.96	1.00	--------	560.00	895.96	23	39	75.00	259.96	1.00	------	300.00	560.00	1,195.96	10	22
Mississippi	282.00	157.15	------	--------	560.00	999.15	12	19	552.00	157.15	------	13.00	--------	560.00	1,282.15	6	6
East Central:																	
Ohio	284.90	------	2.00	400.00	400.00	1,086.90	7	4	284.90	------	2.00	30.00	400.00	400.00	1,116.90	12	8
Indiana	215.00	163.83	------	--------	320.00	698.83	41	45	215.00	163.83	------	24.00	--------	320.00	722.83	42	46
Illinois	640.00	63.26	------	--------	400.00	1,103.26	5	5	640.00	63.26	------	------	--------	400.00	1,103.26	14	14
Michigan	296.40	------	------	--------	480.00	776.40	32	24	296.40	------	------	------	120.00	480.00	896.40	30	23
Wisconsin	485.00	------	------	--------	480.00	965.00	17	10	485.00	------	------	20.00	--------	480.00	985.00	23	18
Minnesota	360.00	------	.50	--------	400.00	760.50	37	26	360.00	------	.50	7.50	--------	400.00	768.00	39	33
Iowa	495.00	------	------	--------	480.00	975.00	16	9	495.00	------	------	5.00	--------	480.00	980.00	24	19
Missouri	307.00	72.38	------	--------	240.00	619.38	46	44	307.00	72.38	------	25.00	--------	240.00	644.38	45	45
Southwestern:																	
Arkansas	205.00	63.96	------	--------	520.00	788.96	31	31	205.00	63.96	------	------	--------	520.00	788.96	36	37
Louisiana	260.00	------	------	--------	560.00	820.00	26	21	520.00	------	------	10.00	--------	560.00	1,090.00	16	12
Oklahoma	346.00	------	------	--------	520.00	866.00	25	16	346.00	------	------	4.50	--------	520.00	870.50	32	26
Texas	271.00	90.00	------	--------	400.00	761.00	36	34	271.00	90.00	------	11.00	--------	400.00	772.00	38	39
West Central:																	
North Dakota	420.00	------	135.00	--------	480.00	1,035.00	10	6	420.00	------	135.00	55.00	--------	480.00	1,090.00	17	13
South Dakota	183.50	------	400.00	--------	400.00	983.50	15	8	183.50	------	400.00	10.00	--------	400.00	993.50	22	17
Nebraska	441.00	124.14	------	--------	480.00	1,045.14	9	12	441.00	124.14	------	30.00	--------	480.00	1,075.14	20	21
Kansas	320.00	176.05	10.00	--------	400.00	906.05	22	30	320.00	176.05	------	10.00	--------	400.00	906.05	29	36
Mountain:																	
Montana	100.00	298.10	------	--------	560.00	958.10	19	35	100.00	298.10	------	130.00	--------	560.00	1,088.10	18	32
Idaho	102.00	------	------	532.00	480.00	1,114.00	4	3	102.00	------	------	------	532.00	480.00	1,114.00	13	9
Wyoming	80.00	129.97	5.00	340.80	400.00	955.77	20	20	80.00	129.97	------	5.00	340.80	400.00	955.77	26	28
Colorado	22.50	129.97	1.00	837.44	480.00	1,470.91	1	2	22.50	129.97	1.00	------	837.44	480.00	1,470.91	2	3
New Mexico	176.50	------	------	--------	480.00	656.50	43	37	176.50	------	------	------	--------	480.00	656.50	44	40
Arizona	106.80	146.60	------	--------	400.00	653.40	44	48	106.80	146.60	------	600.00	--------	400.00	1,253.40	8	11
Utah	205.00	150.69	------	--------	400.00	755.69	39	42	205.00	150.69	------	------	--------	400.00	755.69	40	43
Nevada	76.00	243.75	299.80	--------	480.00	1,099.55	6	18	76.00	243.75	------	299.80	--------	480.00	1,099.55	15	27
Pacific:																	
Washington	160.00	------	83.20	--------	520.00	763.20	34	25	160.00	------	83.20	36.00	--------	520.00	799.20	34	30
Oregon	107.10	------	------	760.00	480.00	1,347.10	2	1	107.10	------	------	------	760.00	480.00	1,347.10	3	2
California	183.00	------	127.00	--------	480.00	790.00	30	23	183.00	------	127.00	724.33	--------	480.00	1,514.33	1	1

40,000-POUND THREE-AXLE TRACTOR-SEMITRAILER (No. 7)

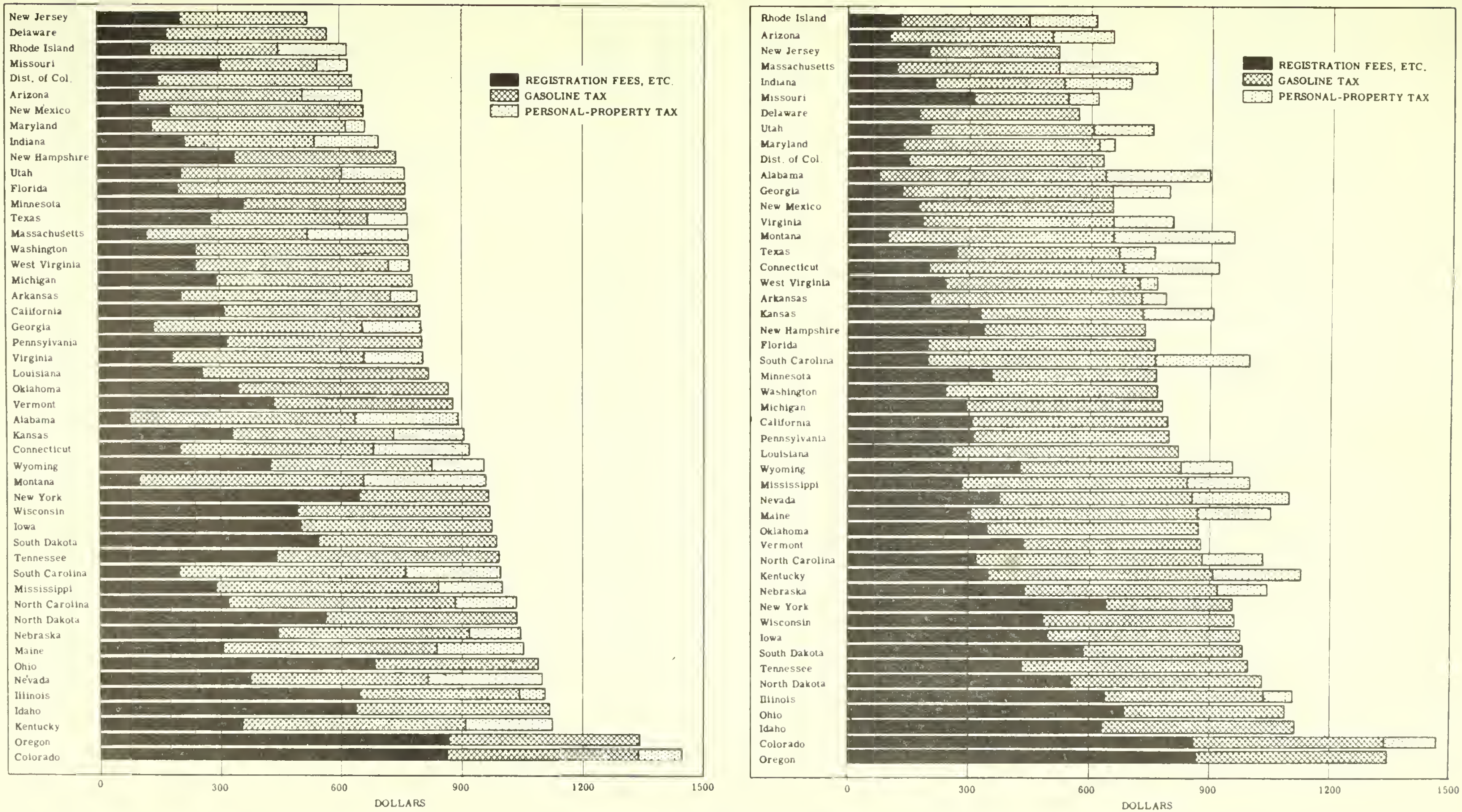


Figure 16.—State road-user and personal-property taxes on a 40,000-pound three-axle tractor-semitrailer combination (No. 7) in private use, ranked according to total taxes (left) and road-user taxes (right).

Table 11.—Road-user and personal-property taxes on a gasoline-powered, four-axle, tractor-semitrailer combination, 50,000 pounds gross vehicle weight (No. 8)

State	Private operation								Contract carrier								
	Registration fee	Property tax	Other taxes and fees	Mileage or ton-mile tax	Gasoline tax	Total	Rank of State		Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Mileage or ton-mile tax	Gasoline tax	Total	Rank of State	
							Total fees and taxes	Total, excluding property tax								Total fees and taxes	Total, excluding property tax
New England:																	
Maine	$355.00	$219.54	------	--------	$1,050.00	$1,624.54	10	17	$355.00	$219.54	------	$30.00	--------	$1,050.00	$1,654.54	18	25
New Hampshire	300.00	------	$116.42	--------	750.00	1,166.42	39	33	300.00	------	$116.42	2.00	--------	750.00	1,168.42	40	36
Vermont	590.00	------	------	--------	825.00	1,415.00	23	16	590.00	------	------	------	--------	825.00	1,415.00	30	27
Massachusetts	152.00	286.00	------	--------	750.00	1,188.00	38	44	152.00	286.00	------	15.00	--------	750.00	1,203.00	38	45
Rhode Island	155.00	200.51	------	--------	600.00	955.51	47	48	155.00	200.51	------	5.00	--------	600.00	960.51	47	48
Connecticut	250.00	240.92	------	--------	900.00	1,390.92	26	34	250.00	240.92	------	5.00	--------	900.00	1,395.92	32	37
Middle Atlantic:																	
New York	298.50	------	------	$816.00	600.00	1,714.50	6	6	298.50	------	------	------	$816.00	600.00	1,714.50	15	14
New Jersey	280.00	------	2.00	--------	600.00	882.00	48	45	280.00	------	2.00	------	--------	600.00	882.00	48	47
Pennsylvania	350.00	------	------	--------	900.00	1,250.00	34	29	350.00	------	------	------	--------	900.00	1,250.00	36	35
Delaware	219.00	------	------	--------	750.00	969.00	46	42	219.00	------	------	------	--------	750.00	969.00	46	44
Maryland	210.00	55.00	------	--------	900.00	1,165.00	40	37	210.00	55.00	------	------	--------	900.00	1,165.00	41	38
District of Columbia	170.00	------	2.00	--------	900.00	1,072.00	43	40	170.00	------	2.00	------	--------	900 00	1,072.00	44	41
West Virginia	317.00	107.65	------	--------	900.00	1,324.65	29	31	317.00	107.65	------	39.50	--------	900.00	1,364.15	34	34
Southeastern:																	
Virginia	300.00	170.74	------	--------	900.00	1,370.74	27	32	300.00	170.74	------	980.00	--------	900.00	2,350.74	4	4
North Carolina	400.00	166.57	------	--------	1,050.00	1,616.57	11	14	700.00	166.57	------	------	--------	1,050.00	1,916.57	10	9
South Carolina	282.00	317.55	------	--------	1,050.00	1,649.55	7	21	282.00	317.55	------	400.00	--------	1,050.00	2,049.55	7	11
Georgia	135.00	185.55	------	--------	975.00	1,295.55	30	36	262.50	185.55	------	25.00	--------	975.00	1,448.05	28	33
Florida	218.10	------	.50	--------	1,050.00	1,268.60	31	26	255.60	------	.50	.50	300.00	1,050.00	1,606.60	20	16
Kentucky	------	------	------	--------	--------	--------	------	------	------	------	------	------	--------	--------	--------	------	------
Tennessee	525.00	------	------	--------	1,050.00	1,575.00	14	7	675.00	------	------	17.50	--------	1,050.00	1.742.50	14	10
Alabama	75.00	310.44	1.00	--------	1.050.00	1,436.44	22	35	75.00	310.44	1.00	------	600.00	1,050.00	2.036.44	8	12
Mississippi	333.00	160.79	------	--------	1,050.00	1,543.79	15	19	654.00	160.79	------	13.00	--------	1,050.00	1,877.79	12	13
East Central:																	
Ohio	338.10	------	2.00	900.00	750.00	1,990.10	4	4	338.10	------	2.00	30.00	900.00	750.00	2,020.10	9	7
Indiana	265.00	208.01	------	--------	600.00	1,073.01	42	47	265.00	208.01	------	24.00	--------	600.00	1,097.01	43	46
Illinois	789.00	75.58	------	--------	750.00	1,614.58	12	8	789.00	75.58	------	------	--------	750.00	1,614.58	19	18
Michigan	333.45	------	------	--------	900.00	1,233.45	35	30	333.45	------	------	------	180.00	900.00	1,413.45	31	28
Wisconsin	620.00	------	------	--------	900.00	1,520.00	17	9	620.00	------	------	20.00	--------	900.00	1,540.00	23	17
Minnesota	510.00	------	.50	--------	750.00	1,260.50	33	28	510.00	------	.50	7.50	--------	750.00	1,268.00	35	32
Iowa	595.00	------	------	--------	900.00	1,495.00	19	11	595.00	------	------	5.00	--------	900.00	1,500.00	25	22
Missouri	507.00	75.89	------	--------	450.00	1,032.89	45	43	507.00	75.89	------	25.00	--------	450.00	1,057.89	45	43
Southwestern:																	
Arkansas	330.00	87.96	------	--------	975.00	1,392.96	25	23	330.00	87.96	------	------	--------	975.00	1,392.96	33	30
Louisiana	320.00	------	------	--------	1,050.00	1,370.00	28	20	640.00	------	------	10.00	--------	1,050.00	1,700.00	16	15
Oklahoma	486.00	------	------	--------	975.00	1,461.00	21	13	486.00	------	------	4.50	--------	975.00	1,465.50	27	23
Texas	336.50	119.80	------	--------	750.00	1,206.30	36	39	336.50	119.80	------	11.00	--------	750.00	1,217.30	37	40
West Central:																	
North Dakota	735.00	------	168.75	--------	900.00	1,803.75	5	5	735.00	------	168.75	55.00	--------	900.00	1,858.75	13	8
South Dakota	183.50	------	565.00	--------	750.00	1,498.50	18	10	183.50	------	------	575.00	--------	750.00	1,508.50	24	21
Nebraska	581.00	165.06	------	--------	900.00	1,646.06	8	12	581.00	165.06	------	30.00	--------	900.00	1,676.06	17	20
Kansas	545.00	186.66	10.00	--------	750.00	1,491.66	20	24	545.00	186.66	------	10.00	--------	750.00	1,491.66	26	31
Mountain:																	
Montana	220.00	356.05	------	--------	1,050.00	1,626.05	9	25	220.00	356.05	------	255.00	--------	1,050.00	1,881.05	11	19
Idaho	102.00	------	------	1,113.00	900.00	2,115.00	3	3	102.00	------	------	------	1,113.00	900.00	2,115.00	6	5
Wyoming	100.00	155.22	5.00	568.73	750.00	1,578.95	13	15	100.00	155.22	------	5.00	568.73	750.00	1,578.95	21	26
Colorado	22.50	155.22	1.00	1,694.16	900.00	2,772.88	1	1	22.50	155.22	1.00	------	1,694.16	900.00	2,772.88	2	2
New Mexico	205.80	------	------	--------	900.00	1,105.80	41	38	205.80	------	------	------	--------	900.00	1,105.80	42	39
Arizona	128.75	174.80	------	--------	750.00	1,053.55	44	46	128.75	174.80	------	1,225.00	--------	750.00	2,278.55	5	6
Utah	280.00	166.55	------	--------	750.00	1,196.55	37	41	280.00	166.55	------	------	--------	750.00	1,196.55	39	42
Nevada	85.50	222.65	332.30	--------	900.00	1,540.45	16	22	85.50	222.65	------	332.30	--------	900.00	1,540.45	22	29
Pacific:																	
Washington	325.00	------	101.50	--------	975.00	1,401.50	24	18	325.00	------	101.50	42.00	--------	975.00	1,443.50	29	24
Oregon	120.40	------	------	1,590.00	900.00	2,610.40	2	2	120.40	------	------	------	1,590.00	900.00	2,610.40	3	3
California	210.00	------	156.00	--------	900.00	1,266.00	32	27	210.00	------	156.00	1,527.83	--------	900.00	2,793.83	1	1

50,000-POUND GASOLINE-POWERED FOUR-AXLE TRACTOR-SEMITRAILER (No. 8)

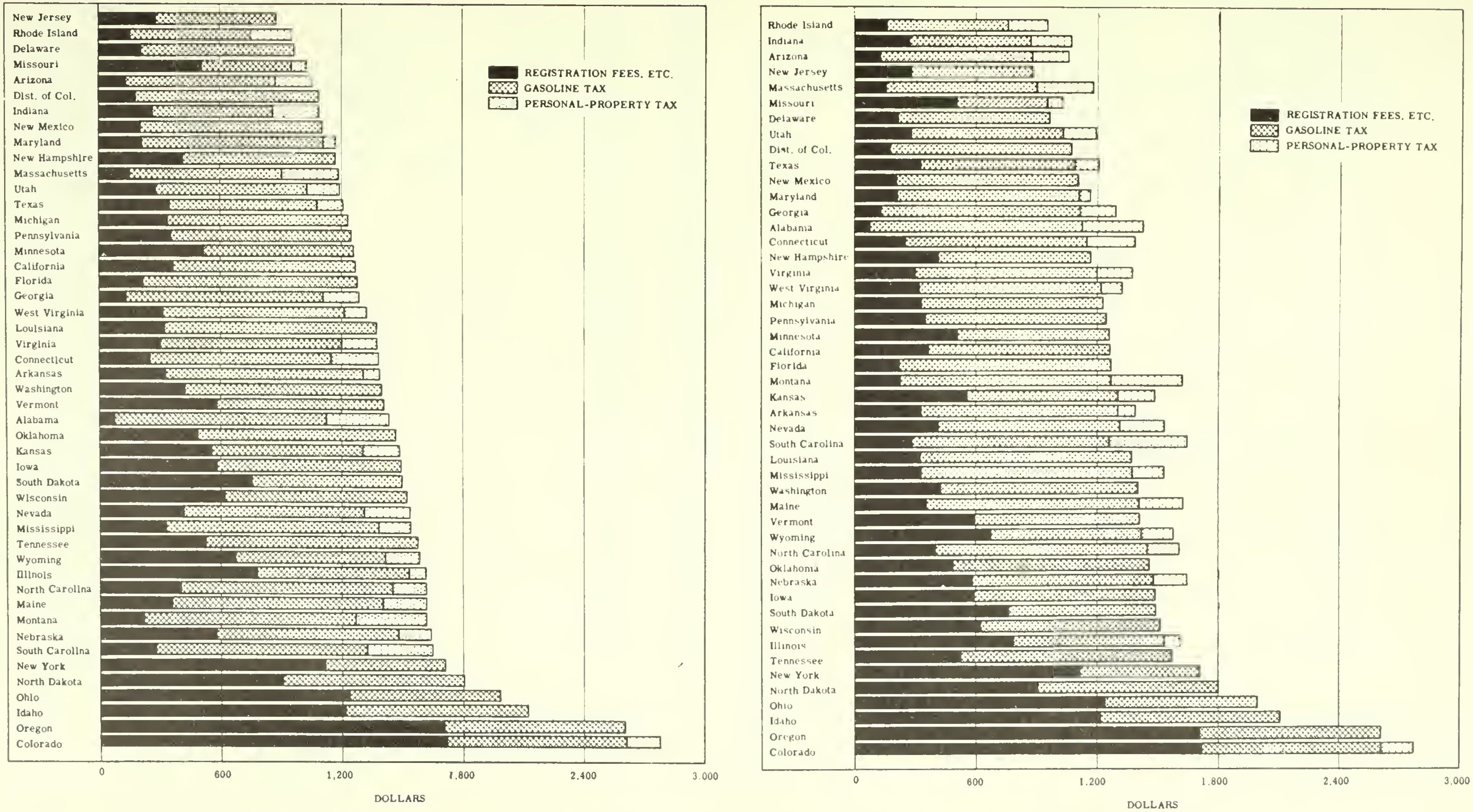


Figure 17.—State road-user and personal-property taxes on a 50,000-pound gasoline-powered four-axle tractor-semitrailer combination (No. 8) in private use, ranked according to total taxes (left) *and road-user taxes* (right).

Table 12.—Road-user and personal-property taxes on a diesel-powered, four-axle, tractor-semitrailer combination, 50,000 pounds gross vehicle weight (No. 9)

State	Private operation								Contract carrier								
							Rank of State									Rank of State	
	Registration fee	Property tax	Other taxes and fees	Mileage or ton-mile tax	Diesel fuel tax	Total	Total fees and taxes	Total, excluding property tax	Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Mileage or ton-mile tax	Diesel fuel tax	Total	Total fees and taxes	Total, excluding property tax
New England:																	
Maine	$355.00	$317.81	------	--------	$700.00	$1,372.81	12	23	$355.00	$317.81	------	$30.00	--------	$700.00	$1,402.81	18	29
New Hampshire	300.00	------	$170.59	--------	500.00	970.59	37	28	300.00	------	$170.59	2.00	--------	500.00	972.59	40	35
Vermont	1,165.00	------	------	--------	--------	1,165.00	24	13	1,165.00	------	------	------	--------	--------	1,165.00	31	25
Massachusetts	152.00	419.00	------	--------	500.00	1,671.00	30	47	152.00	419.00	------	15.00	--------	500.00	1,086.00	34	47
Rhode Island	155.00	293.81	------	--------	400.00	848.81	45	48	155.00	293.81	------	5.00	--------	400.00	853.81	45	48
Connecticut	250.00	411.16	------	--------	600.00	1,261.16	19	36	250.00	411.16	------	5.00	--------	600.00	1,266.16	27	39
Middle Atlantic:																	
New York	300.00	------	------	$816.00	600.00	1,716.00	5	5	300.00	------	------	------	$816.00	600.00	1,716.00	10	8
New Jersey	280.00	------	2.00	--------	400.00	682.00	48	44	280.00	------	2.00	------	--------	400.00	682.00	48	46
Pennsylvania	350.00	------	------	--------	600.00	950.00	39	31	350.00	------	------	------	--------	600.00	950.00	41	37
Delaware	219.00	------	------	--------	500.00	719.00	47	43	219.00	------	------	------	--------	500.00	719.00	47	44
Maryland	210.00	55.00	------	--------	600.00	865.00	43	39	210.00	55.00	------	------	--------	600.00	865.00	43	42
District of Columbia	214.00	------	2.00	--------	600.00	816.00	46	38	214.00	------	2.00	------	--------	600.00	816.00	46	41
West Virginia	317.00	106.20	------	--------	600.00	1,023.20	33	33	317.00	106.20	------	39.50	--------	600.00	1,062.70	36	36
Southeastern:																	
Virginia	300.00	250.21	------	--------	600.00	1,150.21	26	34	300.00	250.21	------	910.00	--------	600.00	2,090.21	5	5
North Carolina	400.00	226.73	------	--------	700.00	1,326.73	15	21	700.00	226.73	------	------	--------	700.00	1,626.73	13	11
South Carolina	262.00	362.92	------	--------	700.00	1,324.92	16	30	262.00	362.92	------	350.00	--------	700.00	1,674.92	11	17
Georgia	190.00	240.50	------	--------	650.00	1,080.50	29	37	372.50	240.50	------	25.00	--------	650.00	1,288.00	26	31
Florida	246.10	------	.50	--------	700.00	946.60	40	32	291.10	------	.50	.50	300.00	700.00	1,292.10	25	19
Kentucky	------	------	------	--------	--------	--------	------	------	------	------	------	------	--------	--------	--------	------	------
Tennessee	525.00	------	------	--------	700.00	1,225.00	21	11	675.00	------	------	17.50	--------	700.00	1,392.50	19	12
Alabama	75.00	455.04	1.00	--------	700.00	1,231.04	20	42	75.00	455.04	1.00	------	600.00	700.00	1,831.04	7	13
Mississippi	333.00	160.79	------	--------	800.00	1,293.79	18	18	654.00	160.79	------	13.00	--------	800.00	1,627.79	12	10
East Central:																	
Ohio	391.30	------	2.00	900.00	500.00	1,793.30	4	4	391.30	------	2.00	30.00	900.00	500.00	1,823.30	8	6
Indiana	265.00	316.19	------	--------	400.00	981.19	36	46	265.00	316.19	------	24.00	--------	400.00	1,005.19	39	45
Illinois	789.00	122.48	------	--------	500.00	1,411.48	10	9	789.00	122.48	------	------	--------	500.00	1,411.48	17	20
Michigan	370.50	------	------	--------	600.00	970.50	38	29	370.50	------	------	------	180.00	600.00	1,150.50	32	27
Wisconsin	620.00	------	------	--------	600.00	1,220.00	22	12	620.00	------	------	20.00	--------	600.00	1,240.00	28	22
Minnesota	510.00	------	.50	--------	500.00	1,010.50	35	25	510.00	------	.50	7.50	--------	500.00	1,018.00	38	32
Iowa	595.00	------	------	--------	700.00	1,295.00	17	8	595.00	------	------	5.00	--------	700.00	1,300.00	24	18
Missouri	507.00	100.41	------	--------	300.00	907.41	42	40	507.00	100.41	------	25.00	--------	300.00	932.41	42	40
Southwestern:																	
Arkansas	330.00	87.96	------	--------	650.00	1,067.96	31	27	330.00	87.96	------	------	--------	650.00	1,067.96	35	34
Louisiana	320.00	------	------	--------	700.00	1,020.00	34	24	640.00	------	------	10.00	--------	700.00	1,350.00	23	16
Oklahoma	476.00	------	------	--------	650.00	1,126.00	28	19	476.00	------	------	4.50	--------	650.00	1,130.50	33	28
Texas	352.60	163.67	------	--------	650.00	1,166.27	23	26	352.60	163.67	------	11.00	--------	650.00	1,177.27	30	33
West Central:																	
North Dakota	735.00	------	168.75	--------	600.00	1,503.75	8	6	735.00	------	168.75	55.00	--------	600.00	1,558.75	14	9
South Dakota	294.50	------	565.00	--------	500.00	1,359.50	13	7	294.50	------	------	575.00	--------	500.00	1,369.50	22	14
Nebraska	546.00	211.92	------	--------	600.00	1,357.92	14	16	546.00	211.92	------	30.00	--------	600.00	1,387.92	21	24
Kansas	545.00	265.78	10.00	--------	700.00	1,520.78	7	10	545.00	265.78	------	10.00	--------	700.00	1,520.78	15	21
Mountain:																	
Montana	220.00	521.76	------	--------	900.00	1,641.76	6	20	220.00	521.76	------	245.00	--------	900.00	1,886.76	6	15
Idaho	102.00	------	------	1,113.00	600.00	1,815.00	3	3	102.00	------	------	------	1,113.00	600.00	1,815.00	9	7
Wyoming	120.00	227.46	5.00	639.00	400.00	1,391.46	11	14	120.00	227.46	------	5.00	639.00	400.00	1,391.46	20	26
Colorado	22.50	227.46	1.00	1,665.20	600.00	2,516.16	2	2	22.50	227.46	1.00	------	1,665.20	600.00	2,516.16	3	3
New Mexico	256.50	------	------	--------	600.00	856.50	44	35	256.50	------	------	------	--------	600.00	856.50	44	38
Arizona	175.50	256.00	------	--------	500.00	931.50	41	45	175.50	256.00	------	1,175.00	--------	500.00	2,106.50	4	4
Utah	280.00	273.26	------	--------	500.00	1,053.26	32	41	280.00	273.26	------	------	--------	500.00	1,053.26	37	43
Nevada	95.50	355.05	369.90	--------	600.00	1,420.45	9	22	95.50	355.05	------	369.90	--------	600.00	1,420.45	16	30
Pacific:																	
Washington	356.25	------	137.30	--------	650.00	1,143.55	27	17	356.25	------	137.30	42.00	--------	650.00	1,185.55	29	23
Oregon	133.70	------	------	2,400.00	--------	2,533.70	1	1	133.70	------	------	------	2,400.00	--------	2,533.70	2	2
California	243.00	------	217.00	--------	700.00	1,160.00	25	15	243.00	------	217.00	1,451.83	--------	700.00	2,611.83	1	1

50,000-POUND DIESEL-POWERED FOUR-AXLE TRACTOR-SEMITRAILER (No. 9)

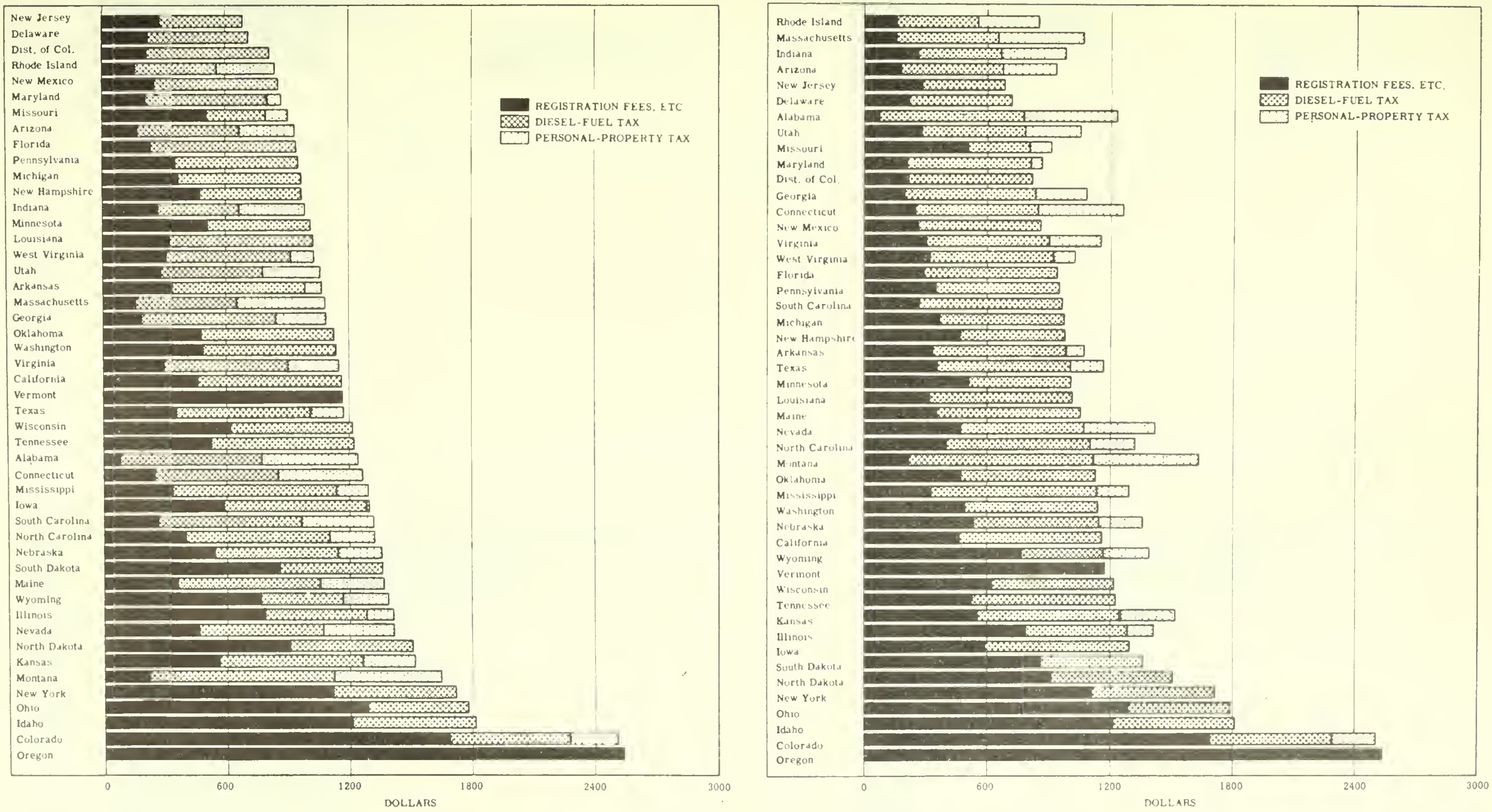


Figure 18.—State road-user and personal-property taxes on a 50,000-pound diesel-powered four-axle tractor-semitrailer combination (No. 9) in private use, ranked according to total taxes (left) and road-user taxes (right).

62,000-POUND DIESEL-POWERED FIVE-AXLE TRACTOR-SEMITRAILER (No. 10)

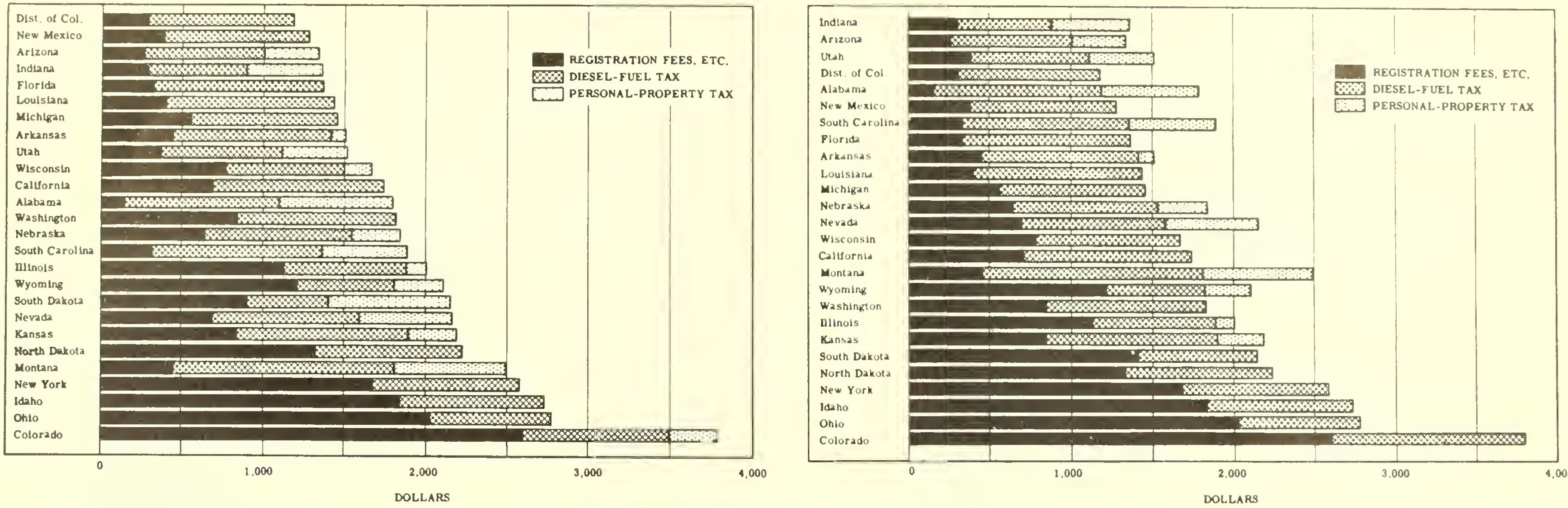


Figure 19.—State road-user and personal-property taxes on a 62,000-pound diesel-powered five-axle tractor-semitrailer combination (No. 10) in private use, ranked according to total taxes (left) *and road-user taxes* (right).

72,000-POUND DIESEL-POWERED FIVE-AXLE TRACTOR-SEMITRAILER AND FULL TRAILER (No. 11)

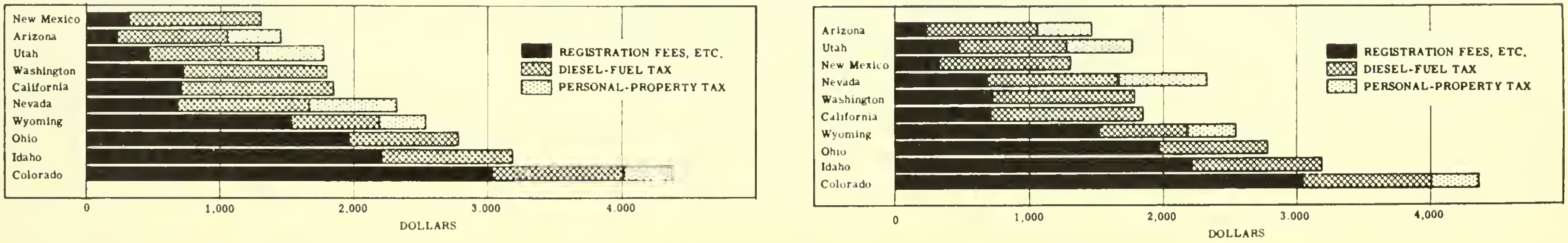


Figure 20.—State road-user and personal-property taxes on a 72,000-pound diesel-powered five-axle tractor-semitrailer and full trailer combination (No. 11) in private use ranked according to total

Table 13.—Road-user and personal-property taxes on heavy combinations (diesel-powered)

State	Private operation								Contract carrier								
							Rank of State									Rank of State	
	Registration fee	Property tax	Other taxes and fees	Mileage or ton-mile tax	Diesel fuel tax	Total	Total fees and taxes	Total, excluding property tax	Registration fee	Property tax	Other taxes and fees	Carrier taxes and fees	Mileage or ton-mile tax	Diesel fuel tax	Total	Total fees and taxes	Total, excluding property tax
FIVE-AXLE TRACTOR-SEMITRAILER COMBINATION, 62,000 POUNDS GROSS VEHICLE WEIGHT (NO. 10)																	
New York	$396.00	------	------	$1,288.00	$893.64	$2,577.64	4	4	$396.00	------	------	------	$1,288.00	$893.64	$2,577.64	8	6
District of Columbia	294.00	------	$2.00	--------	893.64	1,189.64	26	23	294.00	------	$2.00	------	--------	893.64	1,189.64	26	24
South Carolina	327.00	$527.50	------	--------	1,042.58	1,897.08	12	20	327.00	$527.50	------	$400.00	--------	1,042.58	2,297.08	9	17
Florida	329.50	------	.50	--------	1,042.58	1,372.58	22	19	377.00	------	.50	.50	350.00	1,042.58	1,770.58	19	16
Alabama	150.00	603.24	1.00	--------	1,042.58	1,796.82	15	22	150.00	603.24	1.00	------	875.00	1,042.58	2,671.82	7	10
Ohio	628.25	------	2.00	1,400.00	744.70	2,774.95	2	2	628.25	------	2.00	30.00	1,400.00	744.70	2,804.95	5	3
Indiana	300.00	475.53	------	--------	595.76	1,371.29	23	26	300.00	475.53	------	24.00	--------	595.76	1,395.29	24	26
Illinois	1,139.00	127.27	------	--------	744.70	2,010.97	11	8	1,139.00	127.27	------	------	--------	744.70	2,010.97	15	12
Michigan	567.75	------	------	--------	893.64	1,461.39	20	16	567.75	------	------	------	280.00	893.64	1,741.39	20	18
Wisconsin	780.00	------	------	--------	893.64	1,673.64	17	13	780.00	------	------	20.00	--------	893.64	1,693.64	21	19
Arkansas	455.00	87.96	------	--------	968.11	1,511.07	19	18	455.00	87.96	------	------	--------	968.11	1,511.07	23	22
Louisiana	400.00	------	------	--------	1,042.58	1,442.58	21	17	800.00	------	------	10.00	--------	1,042.58	1,852.58	18	14
North Dakota	1,130.00	------	209.25	--------	893.64	2,232.89	6	5	1,130.00	------	209.25	55.00	--------	893.64	2,287.89	10	7
South Dakota	679.50	------	730.00	--------	744.70	2,154.20	9	6	679.50	------	------	740.00	--------	744.70	2,164.20	12	9
Nebraska	651.00	300.39	------	--------	893.64	1,845.03	13	15	651.00	300.39	------	30.00	--------	893.64	1,875.03	16	21
Kansas	845.00	288.73	10.00	--------	1,042.58	2,186.31	7	7	845.00	288.73	------	10.00	--------	1,042.58	2,186.31	11	11
Montana	465.00	691.86	------	--------	1,340.46	2,497.32	5	11	465.00	691.86	------	360.00	--------	1,340.46	2,857.32	4	8
Idaho	102.00	------	------	1,739.50	893.64	2,735.14	3	3	102.00	------	------	------	1,739.50	893.64	2,735.14	6	5
Wyoming	200.00	301.61	5.00	1,011.16	595.76	2,113.53	10	10	200.00	301.61	------	5.00	1,011.16	595.76	2,113.53	14	15
Colorado	22.50	301.61	1.00	2,578.42	893.64	3,797.17	1	1	22.50	301.61	1.00	------	2,578.42	893.64	3,797.17	2	2
New Mexico	387.50	------	------	--------	893.64	1,281.14	25	21	387.50	------	------	------	--------	893.64	1,281.14	25	23
Arizona	264.25	339.20	------	--------	744.70	1,348.15	24	25	264.25	339.20	------	1,750.00	--------	744.70	3,098.15	3	4
Utah	380.00	390.79	------	--------	744.70	1,515.49	18	24	380.00	390.79	------	------	--------	744.70	1,515.49	22	25
Nevada	132.50	565.55	567.90	--------	893.64	2,159.59	8	14	132.50	565.55	------	567.90	--------	893.64	2,159.59	13	20
Washington	662.50	------	193.70	--------	968.11	1,824 31	14	9	662.50	------	193.70	47.00	--------	968.11	1,871.31	17	13
California	423.00	------	283.00	--------	1,042 58	1,748.58	16	12	423.00	------	283.00	2,139.33	--------	1,042.58	3,887.91	1	1
FIVE-AXLE TRACTOR-SEMITRAILER AND FULL TRAILER COMBINATION, 72,000 POUNDS GROSS VEHICLE WEIGHT (NO. 11)																	
Ohio	$557.35	------	$2.00	$1,400.00	$813.95	$2,773.30	3	3	$557.35	------	$2.00	$30.00	$1,400.00	$813.95	$2,803.30	5	5
Idaho	104.00	------	------	2.107.00	976.74	3,187.74	2	2	104.00	------	--------	------	2,107.00	976.74	3.187.74	4	4
Wyoming	170.00	$358.94	7.50	1.352.79	651.16	2.540.39	4	4	170.00	$358.94	--------	7.50	1,352.79	651.16	2.540.39	6	6
Colorado	27.50	358.94	1.00	3,014.46	976.74	4,378.64	1	1	27.50	358.94	1.00	------	3,014.46	976.74	4,378.64	1	2
New Mexico	336.90	------	------	--------	976.74	1,313.64	10	8	336.90	------	------	------	--------	976.74	1,313.64	10	9
Arizona	239.90	403.80	------	--------	813.95	1,457.65	9	10	485.00	403.80	------	1,975.00	--------	813.95	3,677.75	3	3
Utah	470.00	477.31	------	--------	813.95	1,761.26	8	9	470.00	477.31	------	------	--------	813.95	1,761.26	9	10
Nevada	137.50	673.00	542.50	--------	976.74	2,329.74	5	7	137.50	673.00	------	542.50	--------	976.74	2,329.74	7	8
Washington	530,00	------	192.00	--------	1.058.14	1,780.14	7	6	530.00	------	192.00	61.00	--------	1,058.14	1,841.14	8	7
California	364.00	------	346.00	--------	1,139.53	1,849.53	6	5	364.00	------	346.00	2,454.17	--------	1,139.53	4,303.70	2	1

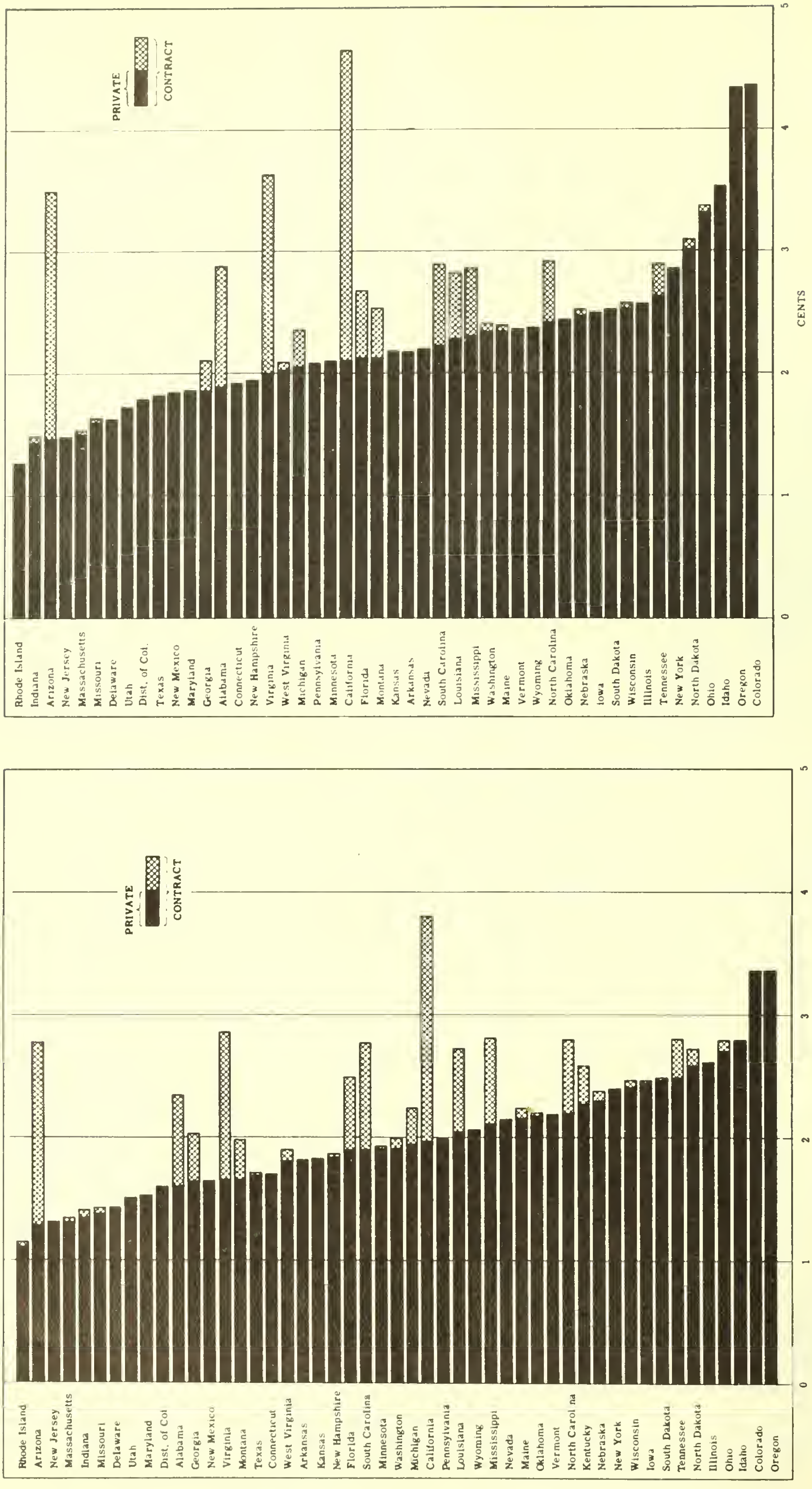


Figure 21.—Comparison of State road-user taxes (personal-property taxes not included), in cents per mile for private use and contract operation, for a 40,000-pound three-axle tractor-semitrailer (No. 7) (left) and a 50,000-pound gasoline-powered four-axle tractor-semitrailer (No. 8) (right).

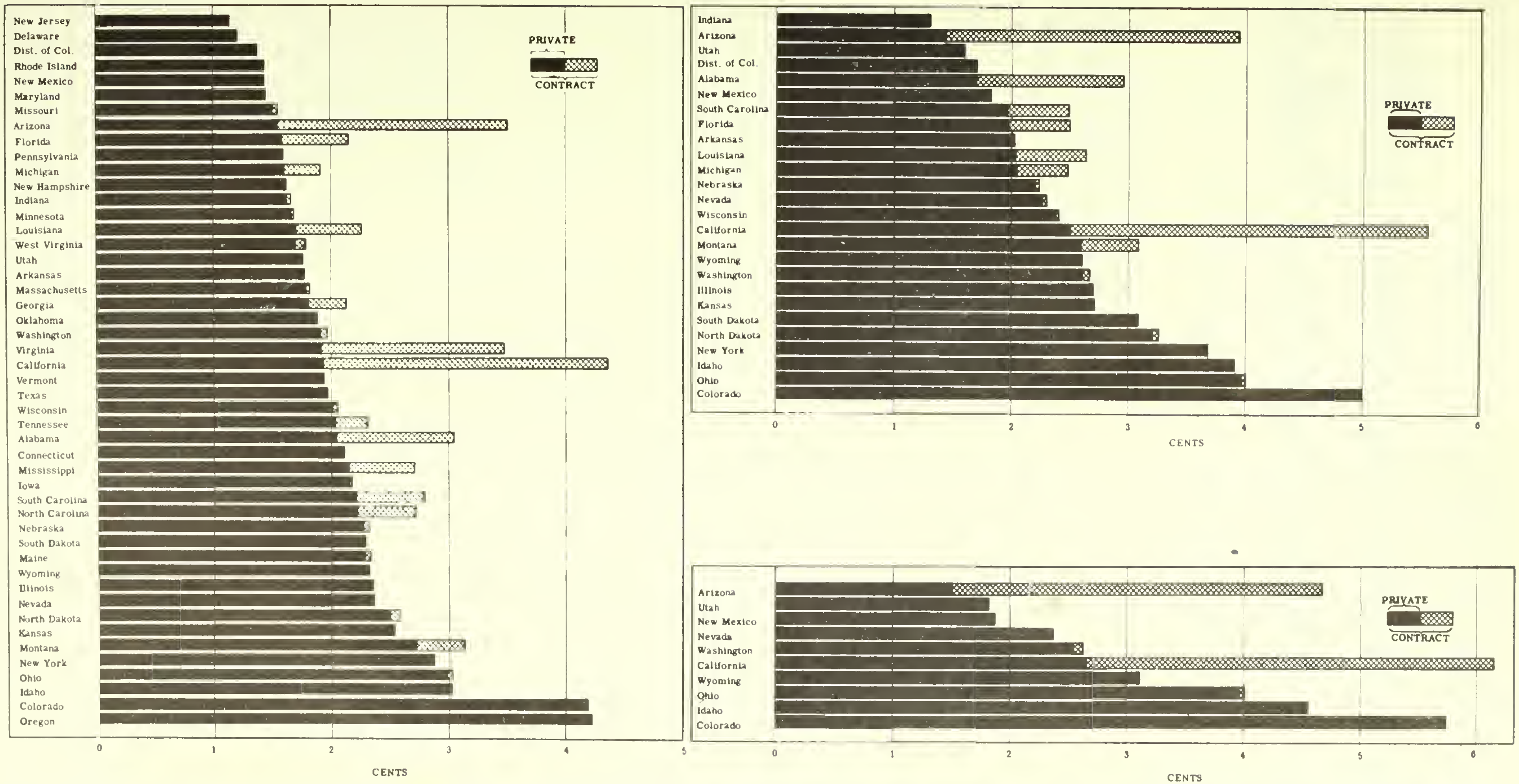


Figure 22.—Comparison of State road-user taxes (personal-property taxes not included), in cents per mile for private use and contract operation, for a 50,000-pound diesel-powered four-axle tractor-semitrailer (No. 9) (left); a 62,000-pound diesel-powered five-axle tractor-semitrailer (No. 10) (upper right); and a 72,000-pound diesel-powered five-axle tractor-semitrailer and full trailer (No. 11) (lower right).

Cost Comparison of Force Account and Contract Construction on Five Secondary Projects in North Carolina

BY THE FINANCIAL AND ADMINISTRATIVE RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported[1] b
FRED B. FARRELL, Chief, Highway Cost Sectio
and MORGAN J. KILPATRICK, Head, Production Cost Un

This article presents the results of comprehensive job cost studies made on three contract and two force account secondary road projects in North Carolina during 1952 and 1953. A complete study of the relative merits of contract and force account methods would require a much wider coverage of kinds and conditions of highway work. Additionally, it would be necessary that social and economic benefits of the two methods be analyzed, that inherent managerial and operational advantages of each method be carefully weighed, and that policy considerations be reviewed. These broader phases of the problem are not part of this study. It is essential that this fact be recognized in appraising the findings presented herein.

This article is published because of its widespread general interest rather than for any current importance it has in North Carolina. The findings of this study may be of considerable value to other countries where the force account method is a more common practice than in the United States.

BOTH the contract and force account methods of doing highway construction and maintenance work have their place in the pattern of highway development. In general, the contract method prevails in construction where standards of materials and performance are specified and controlled, and items of work are planned for construction and measurement. Maintenance work is generally done by the force account method, but there is a definite trend to do a greater percentage by the contract method.

Even though there may be general acceptance of the traditional policy of doing construction by contract, it does not preclude an occasional examination of doing work by the force account method. When judiciously managed, the latter method can be a stabilizing influence in situations where the contract mechanism falls short of meeting its full obligation. Such a situation could be brought about by a rapid increase in volume of highway work accompanied by a shortage of contractors, lack of competition, and rising prices. These were among the factors that concerned the North Carolina State Highway and Public Works Commission after the State undertook its expanded program of secondary road construction in 1949.

This secondary road construction program involved the expenditure of $200 million over a 4-year period. The North Carolina State Highway and Public Works Commission, which has jurisdiction over all rural roads, elected to do a substantial portion of the construction program with its own forces and proceeded to purchase about $5 million of additional road equipment. Legal steps were taken by outside parties, without success, to block such purchases and to require that all construction be done by contract.

Still later, use of Federal-aid secondary funds in constructing some of the secondary projects with State forces was requested. Subsequent arrangements provided that several of these projects would be set up and constructed by either contract or State force methods. It was further agreed that the Bureau of Public Roads would make cost studies on these projects. The number of projects involved was eventually reduced to five by contract and three by State forces. Of these, three contract and two force account projects were selected by the Bureau of Public Roads for detailed unit cost studies during 1952 and 1953. The five projects, totaling about 41 miles in length and costing about $735,000, were situated in central North Carolina.

The purpose of these studies was to develop facts on the comparative costs of the two methods of doing work. It was generally accepted that the usefulness of the study results would be principally in furnishing certain factual background on each method, in minimizing unsubstantiated claims as to excessive economies of one method over the other, and in keeping the main issues involved from being complicated by minor unresolved details.

Pertinent Items

The results presented relate only to the fi projects studied. They should not be viewe as being specifically applicable to all contra and force account construction. The followi items are pertinent:

1. Construction work on each job appeare to reflect normal patterns of operatio There was no evidence of special efforts t make a "good showing."
2. Quality of completed work seemed to b about the same on all jobs.
3. Contractors obtained their aggregat from commercial sources; the State produce most aggregates in its own local quarrie The cost comparison reflects these conditio as they existed. No adjustment was made o the basis of assuming common sources o aggregates or other materials.
4. The several jobs studied were reasonabl typical with respect to terrain and gener nature of the work.
5. Identical cost-keeping procedures we employed on each of the five jobs. All fiel data were obtained by Bureau of Public Road personnel. They kept a daily record of th time spent by each employee and each unit o equipment on each work item. The co tractors and the State made all of their record available to study personnel.
6. For purposes of this cost comparison, uniform schedule of depreciation rates wa applied to each job, contract and force ac count. Separate rates were established fo each major class of equipment. These rate were then applied to the original purchas price for the period that the equipment wa on the job. The depreciation, thus computed was distributed according to working tim spent on each operation performed.
7. In the case of contractors, time and one half was paid any hourly personnel for wor in excess of 40 hours per week. State em ployees paid on an hourly basis did not wor any hours over the scheduled 55 per wee and no overtime was paid for hours worke in excess of 40 per week. The retroactive pa increase granted State employees near th close of this study has not been included i the cost data developed in this article.

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[1] This article was presented at the 35th Annual Meeting of the Highway Research Board, Wash., D. C., Jan. 1956.

able 1.—Example for a single bid item (grading on a mileage basis) showing typical manner in which labor expense and equipment time were kept by work accounts

Cost element	Work accounts							Total
	Grading on a mileage basis, undistributed	Clearing and grubbing within right-of-way	Common excavation	Rock excavation	Drilling and blasting	Building fill	Maintenance for traffic	
LABOR EXPENSE								
Direct wages	$2,492.23	$4,254.19	$1,053.40		$673.38	$616.10	$163.11	$9,252.41
Equipment operation	67.64	561.96	5,608.91	$442.29	156.15	227.24	259.37	7,323.56
Equipment maintenance and repair wages	27.90	89.83	1,553.60	91.83	88.10	37.86	58.89	1,948.01
Liability and compensation insurance	104.69	305.60	311.16	17.97	46.74	48.59	19.23	853.98
Social security payments	64.80	122.85	205.73	13.38	22.98	22.07	12.05	463.86
Leave and holiday charges	72.72	137.86	230.87	15.01	25.79	24.76	13.53	520.54
Total	2,829.98	5,472.29	8,963.67	580.48	1,013.14	976.62	526.18	20,362.36
EQUIPMENT TIME—NET WORKING HOURS								
Crawler tractors		74.1	573.9	161.1	29.3	67.1	7.1	912.6
Scraper, self propelled	9.8		557.7	20.0	5.5		3.3	596.3
Motor graders			992.3		11.6	12.4	96.6	1,112.9
Shovels, draglines, and backhoes	1.3	67.3	166.9					235.5
Dump trucks	9.5	161.9	181.0		13.2			365.6
Industrial rubber-tired tractor					4.7	10.0		14.7
Rollers						13.3		13.3
Compressors			7.7		121.8	13.7		143.2
Loaders			12.5					12.5

8. Any expense on a unit of equipment dur-g its retention on the job was handled as a roject cost.

9. Final costs which were developed for oth the State and the contractors include an lowance for interest on invested capital. o allowance was made for any profit on the ontract. The cost figures, of course, contain ach job's on-project overhead and a pro rata are of off-project overhead expense, includ-g salaries and expenses of management ersonnel. (A $7,500 annual salary rate was lowed for the owners of unincorporated ntractor organizations in determining man-gement expense.)

Bid Items and Work Accounts

The principal items involved in the esti-ates, bids, and payments on the cost study ojects were as follows:

Bid item	*Unit*
rading on a mileage basis	Lump sum cost per mile.
rainage ditch excavation	Cubic yards (outside of the typical roadway section).
xcavation, borrow	Cubic yards.
ase course	Cubic yards, in place.
ituminous surface treatment.	Square yards.
ulvert pipe	Lineal feet, by individual sizes of each type.

At the outset of the cost study, it was de-rmined that additional breakdowns would be eded for several of the bid items, particu-rly in those cases where an item was bid as lump sum or where certain variables, such haul distance, might account for measur-ble differences in costs between jobs. Each d item was therefore subdivided to the ex-nt necessary to show these differences. hese breakdowns are called "work accounts."

For example, one of the bid items on each the jobs was "grading on a mileage basis." his particular item was, in effect, a lump sum st per mile, and included clearing and grub-ng. There was also some clearing and grubbing for borrow pits, material pits, drainage ditches, and other areas outside the typical cross section that was handled by separate bid items. For the purpose of the cost study, therefore, this lump sum grading item was broken down into several work accounts as shown in table 1.

Table 1 is not complete; it simply illustrates the manner in which the work accounts were set up. The upper half of the table shows typical entries for the labor expense elements. These entries were processed from the daily time records kept by study personnel for each man who worked on the grading item at any time during the course of the job. The lower half of the table shows the working time, by classes of equipment, summarized from daily time records kept on each individual unit of equipment. Similar breakdowns were made showing charges to each work account for materials, supplies, services, taxes, depreciation, and so on.

Study personnel also made field measurements of the quantities of work performed on each work account. In the case of grading on a mileage basis, measurements were made of clearing and grubbing, cubic yards of common excavation, and cubic yards of rock excavation involved in this one item. By such means, the costs and corresponding quantities for any work account could be segregated or grouped in various ways to show the principal cost differences between jobs.

To determine the relative costs of doing work by either of the two construction methods, contract or force account, a common base of reference was established. This was done by setting up a hypothetical composite job consisting of typical quantities for the principal work accounts. The relative costs of construction were then determined by applying average unit costs for the two methods to the quantities in the composite job.

Composite Job Cost, Unadjusted

The assumed quantities for the principal work accounts of the composite job are as follows:

Work account	*Quantity*
Clearing and grubbing	20 acres.
Excavation	85,000 cubic yards.
Base course:	
Material	15,000 cubic yards.
Hauling	300,000 ton-miles.
Manipulation	80,000 square yards.
Bituminous surface treatment	80,000 square yards.
15-inch plain concrete pipe	1,000 lineal feet.
18-inch plain concrete pipe	1,500 lineal feet.
24-inch plain concrete pipe	300 lineal feet.

The above rounded quantities account for over 90 percent of the work on a typical secondary road project of the types studied. Arithmetic averages of the final unit costs for each work account, shown separately for the State and contract jobs, are given in table 2.

Application of these unit costs to the quantities for the composite job gives the following totals:

Total cost of composite job, using State's average unit costs	$139,905
Total cost of composite job, using contractors' average unit costs, and excluding profit	134,572

Because of the differing unit costs on the individual jobs, the difference of about 4 percent in favor of contract work is not wholly conclusive. For example, when the separate unit costs for each of the five study projects are applied to the quantities for the composite job, it is found that the total cost of the State force jobs was $128,400 and $151,700, and the contract jobs ranged between $131,600 and $137,800.

Composite Job Cost, Adjusted

The unit costs listed in table 2 represent actual study results, unadjusted for differing job conditions and for certain items of expense,

Table 2.—Average unit costs for State and contract work for work accounts included in the composite job

Work account		Unit cost [1]	
		State jobs	Contract jobs
Clearing and grubbing	acre	$687.00	$391.00
Excavation	cubic yard	.368	.360
Base course:			
Materials [2]	do	2.13	1.87
Hauling	ton-mile	.074	.069
Manipulation	square yard	.095	.106
Bituminous surface treatment	do	.292	.367
15-inch plain concrete pipe	lineal foot	2.20	2.11
18-inch plain concrete pipe	do	4.05	3.78
24-inch plain concrete pipe	do	5.00	5.94

[1] Arithmetic averages. [2] On trucks at loading site.

Fill being built on new location to bridge abutment in background.

Roller used for compacting sand base.

such as taxes, which are borne in varying amounts on force account and contract work. Certain of these conditions and items of expense can be readily evaluated, whereas others can only be approximated. For example, the tax on gasoline can be precisely determined as can the amount of the wage differential between the jobs. In the latter case, however, there is no direct method of measuring the effect of the wage differential on labor productivity.

There is the problem, too, of the compounding effect of a series of adjustments. An adjustment in dollar amounts of one account, such as that involved in equalizing haul distances, will affect dollar amounts of such cost elements as tax payments and interest on the investment. No attempt has been made in this article to analyze these interrelationships. Rather, each adjustment has been computed separately so as to show its particular total dollar amount that is included in the original unadjusted total cost of the composite job.

Included in the costs of the composite job are certain taxes, fees, assessments, and contributions which are incurred as direct and indirect expenses to varying degrees on State force and contract work. Theoretically these items may be viewed as costs which are borne, directly or indirectly, regardless of how the work is performed, whether by State forces or by contract. Hence, it is appropriate to make a comparison with all such determinable costs excluded. For this purpose, the cost e ments involved are divided into two groups

Group I items

In Group I are those items of direct expe involving compulsory payments in the form taxes, fees, and assessments. The rates a amounts of the payments will vary dependi upon whether they are applicable to fo account work or to contract work. Grou covers the following items:

Taxes.—Sales, use, gasoline (State a Federal), lubricating oil, tires and tub automotive equipment and parts, proper communications, corporation franchise, tangibles, project, transportation, and incon

Licenses, fees, and payments.—Labor l bility and compensation, social security, p formance surety bond, project plans, licensi board fee, bidding license, and motor-vehi license.

In the evaluation of these items the vario job costs were analyzed to the extent necessa to depict the actual differences involved. F example, the State crushed most of its agg gate with benefit of tax exemption. On t contract jobs, therefore, a study was made all payments to commercial producers aggregate to determine the amounts of ta and fees therein. Likewise, hauling of ba course and other materials was performed the State with the benefit of certain tax emptions. Accordingly, investigations we made of all payments by the State and t contractor to hired haulers, except railroa to determine the amount of taxes and fe involved. Rental payments for equipme were similarly checked.

The amount of the Group I items rang from 0.2 to 0.9 percent of the total cost on t two State jobs and from 7.9 to 11.3 percent the three contract jobs. Table 3 shows t percentages applicable to the individual wo accounts which comprise the composite job.

Total costs of the Group I items in t composite job, obtained by application of t respective percentages shown in table 3 to t cost of each work account, are as follows:

Using State's costs	$9
Using contractors' costs	12,8

With Group I items excluded, the appare cost advantage is substantially in favor of t contractors' operations. On an individual j basis, each of the three contract jobs shows lower cost than for either of the two State jo

Table 3.—Group I (taxes, fees, and assessments) and Group II (insurance and labor benefit items expressed as a percentage of the total cost of individual work accounts in t composite job

Work account	Group I items [1]		Group II items [1]	
	State jobs	Contract jobs	State jobs	Contract jobs
	Percent	*Percent*	*Percent*	*Percent*
Clearing and grubbing	0.35	7.43	2.51	2.17
Excavation	.43	5.53	4.18	1.37
Base course:				
Materials	.17	11.07	1.87	1.09
Hauling	1.41	14.63	2.21	2.27
Manipulation	1.61	8.44	3.93	2.05
Bituminous surface treatment	.56	11.19	1.37	1.43
15-inch plain concrete pipe	.16	4.42	1.54	1.18
18-inch plain concrete pipe	1.34	5.27	1.96	1.44
24-inch plain concrete pipe	1.34	4.79	1.51	1.25

[1] Arithmetic averages.

Spreading mat stone over asphalt.

Applying asphalt to seal stone, followed by trucks pulling drag brooms.

Group II items

Included in Group II are those items of expense involving optional or discretionary payments in the form of insurance and labor benefits. These differ from the items in Group I in that they fall, to some extent, within the purview of the State or contractor as a matter of management policy. Following are the items included in Group II: (1) Public liability payments or insurance premiums; (2) contributions to employee retirement funds; and (3) compensation for annual leave, sick leave, and holidays.

The State does not carry public liability insurance, but expenses are nevertheless incurred through cash payments for liability claims. There is a statutory limit of litigious liability in the case of the State of $8,000 per claim. The contractor, on the other hand, has no such protection and is rarely in a position to be self-insured. Thus, he usually elects to obtain sufficient protection from insurance companies.

Expenses to the contractors for items 2 and 3 vary to a marked degree between different organizations and the amounts involved are usually determined by the respective managements. Expenses by the State for items 2 and 3 are established by enabling provisions of current civil service regulations. Thus, in the case of the State, these expenses might not be viewed as optional or discretionary. However, they are so considered herein to enable uniformity of treatment with comparable items by the contractor.

Group II items amount to about 2.2 percent of the total project cost on the State jobs and 1.5 percent on the contract jobs. Much of the difference is due to the lower employer contributions and labor benefits on the contractors' jobs. This difference, however, is more than offset by the higher wages paid on the contractors' jobs. Effects of wage differentials are discussed later.

The percentages of the total cost of particular work accounts in the composite job represented by Group II items are shown in table 3.

Total costs of the Group II items in the composite job obtained by application of these percentages to the cost of each work account, are as follows:

Using State's costs	$3,535
Using contractors' costs	2,087

In this instance, the deduction of Group II items from the costs of the composite job would reduce the margin in favor of the contractors.

Wage rate differential

The average hourly wage rate on the contract jobs was about $1.23. This compares with an average of $0.975 on the State jobs. Since about 21 percent of the cost of the composite job, using contractors' costs, represented wages paid directly by the contractor, the contractors' job costs would have been reduced by about $5,860 by paying at the hourly rate prevailing on the State jobs. Included in this $5,860 is approximately $1,200 representing personal income taxes (State and Federal) that would have been paid out of this wage earning increment.

Certain employer contributions and other benefits to labor are associated with wage earnings. Had the wages been reduced on contractors' work in the amount of the aforementioned $5,860, the corresponding benefits would have been reduced by about $580. This amount is in addition to the $5,860.

In summary, the additional costs borne by the contractors on the composite job which are attributable to the wage rate differential are as follows:

"Take-home" pay	$4,660
Personal income taxes	1,200
Labor benefits	580
Total	6,440

This is a substantial amount the deletion of which favors the contractor. However, the extent to which labor productivity or other job factors might be affected by this hypothetical adjustment for wage rate differential was not determined.

Length of haul

There were certain differences in lengths of haul among the several jobs. On excavation it was determined that actual haul distances averaged about 90 feet more on the contract jobs than on the State jobs. To compensate for this difference, an amount of $0.005 per cubic yard was deducted from the cost of contract work on the composite job. The dollar amount is $425.

For the base course, the average haul by the State and the contractor varied by such a minor distance that no adjustment was warranted.

In the case of bituminous surface treatment, the surface aggregate was hauled by truck from State-owned local quarries to the State jobs; by truck from local commercial sources on two of the contract jobs; and by a combination of rail and truck on the remaining contract job. The longer hauls prevailed on the contract jobs. The cost of hauling this additional distance amounted to about $0.023 per square yard of bituminous surface treatment. The dollar amount of this reduction on the contractors' cost of the composite job is $1,840.

The effect of the foregoing adjustments for haul lengths is to reduce the contractors' cost in each instance. The total reduction in the cost of the composite job, using contractors' costs, is $2,265. This consists of $425 for excavation and $1,840 for surface aggregate.

Base course materials

There were wide differences in practices for procuring base course materials that warrant consideration. On the State jobs, base course materials were obtained from State-owned and operated quarries and pits in the vicinity of the projects. For the contract jobs, crushed rock was obtained from commercial sources on one project and hauled by truck; commercially produced soil was shipped in by rail on another; and on the last contract

Table 4.—Effect of certain cost variables on total cost of composite job

Nature of variable	Amount of variable in composite job [1]		Total cost of composite job, after deducting the amount of the variable		
	State	Contract	State	Contract	Difference, in favor of contractors [2]
Group I items (taxes, fees, and assessments)	$908	$12, 885	$138, 997	$121, 687	$17, 310
Group II items (insurance and labor benefits)	3, 535	2, 087	136, 370	132, 485	3, 885
Wage rate differential	------	6, 440	139, 905	128, 132	11, 773
Haul distance differential	------	2, 265	139, 905	132, 307	7, 598
Base course materials	31, 950	28, 050	107, 955	106, 522	1, 433
Interest on State's investment	2, 385	-------	137, 520	134, 572	2, 948

[1] Amounts are not additive.

[2] Includes the initially computed cost differential of $5,333 in favor of the contractors.

job, soil was obtained by the contractor in local pits adjacent to the project.

There are several alternatives that would have materially affected the cost comparison. For example, the State could have elected to furnish materials to the contractor; on the other hand, the State could have elected to purchase either rock or soil from commercial sources. Any attempt to equalize the varying practices by computing probable costs for any one or a combination of these alternatives would require many assumptions that could not be supported by actual field data.

In view of these variables with respect to the base course item, it is likely that the most valid comparison would be that in which the cost of base course materials is eliminated. For the composite job, the cost of these materials was $31,950 using State's prices and $28,050 using contractors' prices. Elimination of these costs would tend to reduce the margin in favor of the contractors.

Interest on equipment investment

Whereas depreciation of equipment is an expense that must be met regardless of how the work is done, the matter of interest on the investment in equipment is a point on which opinion may differ. For purposes of uniformity, interest charges were computed in similar fashion on both State and contract jobs during the course of the studies. The total costs of interest tended to be somewhat higher on the contract jobs. This was due, in part, to the contractors' practice of keeping equipment on the job for somewhat longer periods of time. (On the State jobs, it was common practice to move idle or standby equipment off the job and put it to use on other nearby construction and maintenance work in the district.) Expressed as a percentage of the total costs of the composite job, interest costs were 1.6 percent on State work and 1.8 percent on contract work. By applying these percentages to the costs of the composite job on State work ($139,905) and on contract work ($134,572) the following amounts are derived for interest on investment in equipment:

Using State's costs	$2, 385
Using contractors' costs	2, 422

It is generally recognized that interest on the investment is a proper cost of contractors' work. From the standpoint of comparative economic cost, it is also a proper cost of State force work. However, it is a cost for which a public agency seeks no return and one which would not be met by actual cash transactions in case of State work. Hence, there is some basis for excluding this item of expense from the State's total costs.

Other variables

There were other variables. Some were of minor importance with respect to their effect on costs; others could not be readily evaluated. A few are as follows:

1. *Maintenance of public traffic.*—For the composite job, this expense amounted to $384 on State work and $320 on contract work.

2. *Moving in and moving out.*—For the composite job, this expense amounted to $1,088 on State work and $1,469 on contract work. Ordinarily it might be expected that this difference would be much greater. However, two of the contractors had permanent headquarters in the immediate vicinity of the projects. The other was about 100 miles distant. The State's expense was increased somewhat by the practice of frequently moving its equipment on and off the job to meet fluctuating requirements on other construction and maintenance work in the district. The contractors did not move their equipment quite so often.

3. *Age of equipment.*—The average age of trucks, for example, was about the same on three of the five jobs studied, one contract and two State jobs. It was about twice as great on one of the other contract jobs. On the third contract job, age data for trucks were not available.

4. *Size of equipment.*—The size of comparable units was about the same on all jobs. There were certain exceptions, such as scraper on the contract jobs which averaged about 3 percent larger than those used by the State.

5. *Major delays.*—Delays of 15 minutes o more in duration with weather excluded wer about the same on both the State and th contract jobs.

6. *Minor delays.*—Delays of less than 1 minutes each in duration were slightly greate on the State jobs.

Cost Comparison Summary of Composite Job

As previously stated, the initial computation of total costs of the composite job shows the following:

Total cost of composite job, using State's average unit costs	$139, 905
Total cost of composite job, using contractors' average unit costs and excluding profit	134, 572
Initially computed cost differential, in favor of contractors	5, 333

The total costs and the differential will vary depending on the adjustments that may be made to show the effect of individual cost elements. The principal adjustments are listed in table 4. In the cases of the Group I items, the wage rate differential, and the haul distance differential, the adjustments result in increases in the original spread of $5,333 between the costs of the composite jobs. In the other three instances listed in the table, the adjustments decrease the original spread but in no case is any one adjustment sufficient to overcome it completely.

With the exception of Group I and II items and interest, the variables listed in table 4 are not wholly additive. Each has been separately computed on the basis of its amount in the initially developed total cost of the composite job. There are, for example, Group I and Group II items involved in adjustments for wage differential, and all three together with interest are involved to a certain extent in the haul distance differential and base course materials.

A more refined analysis could be developed to show the net and cumulative effects of each of these variables and also give an indication of the amount of other cost variables which are not herein itemized or evaluated. Such computations would not, however, be sufficiently weighted in favor of State work to alter the finding that, exclusive of profit, contract work was performed at a lower construction cost than State force work on the five jobs studied. This finding is limited solely to the circumstances herein reported. No attempt was made in this particular study to determine whether or to what extent contract or State force work was the more economic in all phases.

Motor-Vehicle Size and Weight Limits

A comparison of State legal limits of motor-vehicle sizes and weights with standards recommended by the American Association of State Highway Officials is given in a table on pages 54–55. The statutory limits reported in this tabulation, prepared by the Bureau of Public Roads as of July 1, 1956, have been reviewed for accuracy by the appropriate State officials.

Statutory limits are shown for width, height, and length of vehicles; number of towed units; maximum axle loads for single and tandem axles; and maximum gross weights for single-unit trucks, truck-tractor semitrailer combinations, and other combinations.

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Highway Practice in the United States of America (1949). 75 cents.
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1948, 65 cents. 1952, 75 cents.
Highway Statistics, Summary to 1945. 40 cents.
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Mathematical Theory of Vibration in Suspension Bridges (1950). $1.25.
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United States System of Numbered Highways. 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

Bibliography on Automobile Parking in the United States (1946).
Bibliography on Highway Lighting (1937).
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Bibliography on Land Acquisition for Public Roads (1947).
Bibliography on Roadside Control (1949).
Express Highways in the United States: a Bibliography (1945).
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DEPARTMENT OF COMMERCE - BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF JUNE 30, 1956

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES 1/	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$26,442	$14,910	$8,123	212.2	$5,370	$2,863	40.7	$46,628	$24,755	651.5	$66,908	$35,741	904.4
Arizona	14,328	4,288	3,225	72.0	2,356	1,724	34.1	11,321	8,393	101.6	17,965	13,342	207.7
Arkansas	23,973	15,734	7,887	505.9	5,122	2,632	46.5	22,773	11,766	480.2	43,629	22,285	1,032.6
California	67,340	18,614	10,094	199.4	9,747	4,902	9.7	147,987	76,255	247.0	176,348	91,251	456.1
Colorado	27,597	8,408	4,832	157.4	1,078	611	4.6	21,129	11,479	225.0	30,615	16,922	387.0
Connecticut	30,536	4,083	1,956	7.2	2,015	1,125	7.6	12,772	6,437	20.2	18,870	9,518	35.0
Delaware	10,746	3,490	1,749	33.4	1,188	611	14.2	5,685	2,847	49.4	10,363	5,207	97.0
Florida	24,979	6,859	3,629	107.3	13,779	7,101	45.0	37,501	19,425	336.0	58,139	30,155	488.3
Georgia	37,181	32,641	16,596	709.3	13,136	6,774	109.9	47,664	22,631	778.6	93,441	46,001	1,597.8
Idaho	16,652	4,680	3,010	108.3	2,860	1,892	72.9	13,948	9,044	204.9	21,488	13,946	386.1
Illinois	61,677	34,818	18,233	619.1	17,862	9,003	83.8	101,527	54,716	625.4	154,207	81,952	1,328.3
Indiana	48,451	20,917	11,384	90.4	15,951	8,185	169.7	47,981	26,195	206.3	84,849	45,764	466.4
Iowa	27,753	18,859	10,864	757.9	4,959	2,773	78.4	31,020	16,680	896.4	54,838	30,317	1,732.7
Kansas	24,921	11,601	5,811	924.2	9,371	4,841	161.1	28,505	14,787	818.4	49,477	25,439	1,903.7
Kentucky	32,357	6,856	3,462	154.9	1,727	870	4.1	42,184	22,517	614.0	50,767	26,849	773.0
Louisiana	21,594	22,938	11,637	75.5	3,982	1,998	9.0	42,396	20,771	372.2	69,316	34,406	456.7
Maine	11,643	10,522	5,417	89.6	1,468	812	8.7	15,492	7,983	103.8	27,482	14,212	202.1
Maryland	17,009	18,131	9,278	121.6	16,627	7,712	23.6	24,097	12,436	153.6	58,855	29,426	298.8
Massachusetts	33,060	20,293	10,137	22.9	13,378	7,252	8.9	42,310	20,466	62.2	75,981	37,855	94.0
Michigan	45,648	33,172	16,698	656.2	20,707	10,886	95.0	65,158	33,772	544.8	119,037	61,356	1,296.0
Minnesota	29,901	9,132	4,749	580.7	7,182	3,799	63.4	47,656	25,551	1,529.7	63,970	34,099	2,173.8
Mississippi	26,015	11,608	5,733	463.0	5,078	2,889	115.3	25,486	12,907	769.7	42,172	21,529	1,348.0
Missouri	35,681	20,361	10,509	1,103.3	6,553	3,490	12.9	78,428	40,941	1,469.5	105,342	54,940	2,585.7
Montana	26,636	11,553	6,845	203.4	4,301	2,611	54.1	30,042	18,668	542.4	45,896	28,124	799.9
Nebraska	28,930	9,046	4,891	290.6	4,548	2,283	93.3	34,874	18,532	1,037.3	48,468	25,706	1,421.2
Nevada	17,212	6,673	5,569	86.4	708	591	25.1	11,094	9,363	203.2	18,475	15,523	314.7
New Hampshire	11,970	2,633	1,499	16.1	556	304	2.4	10,073	5,157	61.5	13,262	6,960	80.0
New Jersey	39,937	12,629	6,324	54.8	6,972	3,064	17.3	35,351	17,088	45.8	54,952	26,476	117.9
New Mexico	17,042	1,965	1,256	17.4	4,399	2,865	62.5	12,535	8,088	180.6	18,899	12,209	260.5
New York	103,102	25,042	13,040	86.2	56,130	30,505	97.1	234,331	111,018	324.5	315,503	154,563	507.8
North Carolina	35,069	20,031	9,829	315.8	2,964	1,498	68.4	57,819	28,424	780.3	80,814	39,751	1,164.5
North Dakota	16,135	8,388	4,256	1,005.3	10,641	5,566	664.0	13,086	6,669	719.8	32,115	16,491	2,389.1
Ohio	61,392	53,383	27,827	205.9	9,511	4,780	51.2	90,204	45,032	144.1	153,098	77,639	401.2
Oklahoma	29,114	15,495	8,110	288.7	13,874	7,153	183.4	41,017	21,203	392.3	70,386	36,466	864.4
Oregon	17,296	3,805	2,247	71.9	848	507	6.2	29,136	17,912	278.9	33,789	20,666	357.0
Pennsylvania	71,717	44,177	23,418	153.0	22,311	11,135	47.0	127,986	64,072	381.1	194,474	98,625	581.1
Rhode Island	8,218	1,987	994	4.7	1,609	810	1.0	18,964	9,795	28.2	22,560	11,599	33.9
South Carolina	22,012	17,952	9,734	299.9	1,976	1,240	11.0	21,277	11,018	483.2	41,205	21,992	794.1
South Dakota	13,456	15,815	9,203	621.4	7,892	4,567	212.6	18,175	10,364	675.4	41,882	24,134	1,509.4
Tennessee	37,103	16,830	8,130	426.7	7,677	3,841	19.9	49,861	23,089	556.5	74,368	35,060	1,003.1
Texas	73,648	15,577	8,068	345.2	17,310	9,423	124.0	123,379	64,715	1,688.2	156,266	82,206	2,157.4
Utah	11,546	8,065	5,913	172.8	1,442	1,093	15.5	13,335	9,991	147.3	22,842	16,997	335.6
Vermont	10,537	1,263	632	23.6	1,310	654	7.3	10,693	5,514	94.2	13,266	6,800	125.1
Virginia	31,027	17,574	9,302	304.3	7,920	4,158	100.1	33,103	16,477	340.2	58,597	29,937	744.6
Washington	20,714	14,198	7,972	174.9	9,667	5,154	107.1	27,540	14,957	260.4	51,405	28,083	542.4
West Virginia	24,928	12,915	6,664	57.2	7,048	3,628	21.3	16,737	8,422	53.3	36,700	18,714	131.8
Wisconsin	34,042	13,188	6,731	242.0	7,582	3,980	35.7	52,751	25,956	535.5	73,521	36,667	813.2
Wyoming	12,232	4,431	2,921	70.5	2,917	1,846	40.2	18,006	11,853	346.7	25,354	16,620	457.4
Hawaii	3,623	3,416	1,698	9.8	4,101	1,981	3.8	3,972	2,001	4.2	11,489	5,680	17.8
District of Columbia	8,228	13,976	8,019	2.5	2,564	1,265	1.2	7,604	3,785	.7	24,144	13,069	4.4
Puerto Rico	7,425	6,605	2,658	18.7	3,438	1,698	3.6	18,751	8,687	60.4	28,794	13,043	82.7
Alaska	1,933												
TOTAL	1,491,708	731,527	388,763	13,341.4	403,712	212,945	3,295.4	2,121,344	1,100,604	21,626.6	3,256,583	1,702,312	38,263.4

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Northern Circumferential Highway (State Route 128) which skirts the congested areas of Boston, Mass.

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U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretary

BUREAU OF PUBLIC ROADS
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Highway-User Tax Schedules Recommended in State Highway Finance Studies

BY THE FINANCIAL AND ADMINISTRATIVE RESEARCH BRANCH BUREAU OF PUBLIC ROADS

**Reported by G. P. St. CLAIR, Chief of Branch,
and HUGO C. DUZAN, Transportation Economist**

The findings of highway tax studies made in 9 States are compared in this article in terms of the tax payments that would be required of 8 vehicles or vehicle combinations under the recommended user-tax schedules. Comparative data on the payments that would be made under existing tax laws are also given. The payments are expressed in terms of payments per year, payments per mile of travel, and payments per gross ton-mile.

As might be expected, there is considerable variation from State to State in the recommended user-tax payments for each of the 8 vehicles. Nevertheless, there is a marked central tendency in that the study findings, particularly those for heavy vehicles, tend to group within relatively narrow intervals.

The median or "middle" values of the required tax payments per year are $40 for the light passenger car, $1,229 for the 4-axle tractor-semitrailer combination, and $1,836 for the 5-axle tractor-semitrailer-trailer combination. Expressed as indexes of the passenger-car payment, the values for the three vehicles are 1.0, 30.7, and 45.9.

Median tax payments per mile of travel are 0.43 cents for the passenger car, 3.07 cents for the 4-axle tractor-semitrailer combination and 3.67 cents for the tractor-semitrailer-trailer combination. The indexes are 1.0, 7.15, and 8.54.

The trend of median tax payments per gross ton-mile is downward, dropping from 2.2 mills for the passenger car to 1.2 mills for the 4-axle tractor-semitrailer combination and 1 mill for the 5-axle tractor-semitrailer-trailer combination. The median indexes of required tax payments per ton-mile are 1.0, 0.55, and 0.46.

THE findings of highway tax studies made by State personnel or consultants in individual States throw some light on the vexatious question of the relative tax responsibility of vehicles of different sizes and weights. Although studies have been made in numerous States, in only nine of them were the findings expressed in terms such that a comparative analysis could readily be made. These States are California, Colorado, Illinois, Louisiana, Minnesota, New York, Ohio, Utah, and Washington. The tables and charts which follow deal with the study findings in those nine States and with comparative data on actual payments under existing user-tax laws.

Appendix B (p. 83) gives a list of the State highway finance and taxation studies which provided the data for this analysis.

Vehicles Selected for Comparison

In order to run the gamut of motor-vehicle sizes and weights, eight typical vehicles or vehicle combinations shown in silhouette below were selected. These vehicles and their assumed operating conditions are briefly described in table 1.

The maximum gross weights chosen are typical of the weights for which vehicles and combinations of these several types would be registered. Annual mileages also are reasonably close to average. For purposes of State-to-State comparison it is assumed that a vehicle travels its entire mileage in the particular State. It is recognized that over-the-road trucking combinations may divide their actual travel among several States, and also that such vehicles may have much greater total mileages than those used in this comparison. The values chosen, although not averages of those used in the studies, are considered moderate and fair. For uniformity of comparison all of the vehicles were taken as gasoline-powered.

In calculating required tax payments for the eight selected vehicles from the schedules recommended by the investigators, the effort was made to render an accurate interpretation of the findings of the tax studies in each of the nine States. Values were calculated by two methods. First, the required tax payments for each vehicle were computed on the basis of the values of vehicle weight, annual mileage, and miles per gallon used in each State study. These values, which are excluded from the body of the report because they are not truly comparable from State to State, are given in Appendix A (pp. 79–82). The second method, which affords direct State-to-State comparisons, utilizes the values of maximum gross weight, annual travel, and miles per gallon given in table 1. It is believed that the resulting comparisons are accurate and fair; but it is not unlikely that minor differences in interpretation would occur in similar calculations made by others.

It was desired to present a comparison of the tax-study findings in these nine States with the taxes that would have been paid by the same vehicles under the user-tax rates prevailing in 1955. For this comparison two more States were added to the list: Oregon, which has a tax structure based on the findings of incremental studies; and Idaho, which has a mileage tax similar in coverage to those in Colorado, New York, Ohio, and Oregon.

In each of the tables and charts presented the user-tax payments shown for a given vehicle were obtained by adding (*1*) the registration fee or weight tax that the vehicle

No. 1 - Passenger car

No. 2 - Pickup

No. 3 - 2-axle, 6-tire truck

No. 4 - Bus

No. 5 - 3-axle tractor-semitrailer

No. 6 - 4-axle tractor-semitrailer

No. 7 - Truck-trailer

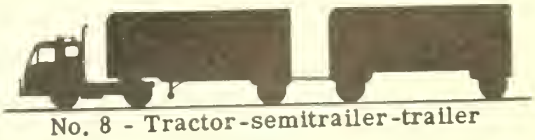
No. 8 - Tractor-semitrailer-trailer

Table 1.—Characteristics of vehicles selected for tax-study comparisons

Vehicle number	General description	Empty weight	Maximum gross weight		Annual travel		Ton-miles at maximum gross weight		Fuel consumption rate [1]
			Pounds	Index	Miles	Index	Ton-miles	Index	
		Pounds							*Miles/gal.*
1	Passenger car	2,982	[2] 3,882	1.00	9,300	1.00	18,051	1.00	16.7
2	Pickup truck	3,266	4,800	1.24	8,000	.86	19,200	1.06	15.6
3	2-axle, 6-tire truck	7,675	19,000	4.89	10,000	1.08	95,000	5.26	7.8
4	Bus	20,565	27,000	6.96	50,000	5.38	675,000	37.39	5.3
5	3-axle tractor-semitrailer	15,215	40,000	10.30	38,000	4.09	760,000	42.10	4.8
6	4-axle tractor-semitrailer	19,050	50,000	12.88	40,000	4.30	1,000,000	55.40	4.2
7	5-axle truck-trailer	27,500	68,000	17.52	50,000	5.38	1,700,000	94.18	3.5
8	5-axle tractor-semitrailer-trailer	27,445	72,000	18.55	50,000	5.38	1,800,000	99.72	3.4

[1] All vehicles were gasoline-powered.
[2] Six passengers at 150 pounds each assumed.

would pay under the tax-study recommendations (or existing law); (*2*) the gasoline tax that would be paid under the assumed annual mileage and miles per gallon; and (*3*) the amount of third-structure tax (such as the New York weight-distance tax or the Ohio axle-mile tax) that would be required. Values are expressed in three ways as follows: (*1*) Tax payment per year, (*2*) tax payment per mile of travel, and (*3*) tax payment per gross ton-mile (obtained by dividing the tax payment by the product of maximum gross weight and annual mileage). The latter is a rather controversial figure, as there is no general agreement that gross ton-miles are a true measure of tax responsibility. Values are also expressed in the form of indexes, the index value for the passenger car being 1.00 in all cases.

Since vehicles do not travel fully loaded all of the time, average operating gross weights might well have been used instead of maximum gross weights to compute the tax payments per ton-mile. Average operating weights, however, are affected by the type of operation and those for vehicles of a given maximum gross weight may vary widely. Maximum gross vehicle weight, which is the registration basis in the majority of States and represents the potential of the vehicle, was therefore chosen.

Median Values of Tax-Study Recommendations

The first question to be asked about these State tax-study findings is, "What is the trend?" As arithmetic averages are likely to be deceptive in such a case, it was decided to compare the median [1] values, from among the findings in these nine States, of the user-tax payments required of each of the eight selected vehicles. These values, which are given in the upper section (part 1) of table 2 and shown graphically in figure 1, are indicative of the middle ground of tax-study findings.

Running quickly over the top line of table 2, we find the middle values of tax-study findings ranging from $40 per year for the passenger car to $207 per year for the 2-axle, 6-tire truck; $1,009 for the bus; $1,229 for the 4-axle, 50,000-pound tractor-semitrailer; and $1,836 for the 5-axle, 72,000-pound tractor-semitrailer-trailer combination. Expressed in terms of index values the range is from 1.00 for the passenger car to 45.90 for the biggest combination.

Expression of the tax-study findings in terms of required tax payment per mile of travel eliminates the effect of the greater mileages traveled by the heavier vehicles, and thus narrows the range of variation. The median values of recommended tax payments per mile are found to vary from 0.43 cents in the case of the passenger car to 3.67 cents for the biggest combination. In terms of index values the range is from 1.00 to 8.54.

By the expression of the tax-study findings in terms of recommended tax payments per gross ton-mile, the trend-line is caused to decrease rather than increase with weight of vehicle. Although a number of the tax studies in these nine States were based on the gross ton-mile theory, which holds that user taxes should be paid in proportion to the product of weight and distance traveled, there was a tendency for the investigators to mitigate their theoretical findings somewhat when faced with the task of devising and recommending an actual schedule of taxation.

The downward trend in payments per ton-mile under tax-study recommendations is shown very plainly in the bottom panel of figure 1. If the gross ton-mile concept were fully accepted this trend would be horizontal.

Median User-Tax Payments at 1955 Rates

The lower portion of table 2 gives the median values, for the same nine States plus Idaho and Oregon, of the user-tax payments required of the eight selected vehicles under the tax schedules prevailing in 1955. The actual values are shown graphically in figure 2.

The median values of actual required tax payments show similar trends to those of the tax-study recommendations, the values being higher in some cases and lower in others. Although the existing tax schedules in most of the nine tax-study States impose lower requirements on the heavy truck combinations than those recommended, the median values fail to reflect this tendency. This is largely due to the inclusion of values for Idaho and Oregon, which were not in the tax-study group. Since the tax schedules in these two States are generally on the high side, the median values are definitely higher than they would have been if the two States had not been included. The median values for the 11 States are, however, indicative of the trend of required tax payments in States that have, in recent years, given definite attention to the problem of allocating user-tax responsibility among vehicles of different sizes and weights. The ranges in required values may be expressed briefly as follows:

	From passenger-car payment of—	*To tractor-semitrailer-trailer payment of—*
Per year	$41	$2,214.
Per mile	0.44 cents	4.43 cents.
Per ton-mile	0.23 cents	0.12 cents.

Table 2.—Median road-user tax payments for selected vehicles

Basis for payment	No. 1: Passenger car	Single-unit trucks		No. 4: Bus	Truck combinations			
					Tractor-semitrailer		No. 7: Truck-trailer	No. 8: Tractor-semi-trailer-trailer
		No. 2: Pickup	No. 3: 2-axle, 6-tire		No. 5: 3-axle	No. 6: 4-axle		
PART 2.—TOTAL USER-TAX PAYMENTS UNDER TAX-STUDY RECOMMENDATIONS IN 9 STATES								
Payments per year:								
Median payments ..... dollars	40	46	207	1,009	965	1,229	1,710	1,836
Index	1.00	1.15	5.18	25.23	24.13	30.73	42.75	45.90
Payments per mile:								
Median payments ..... cents	0.430	0.575	2.070	2.018	2.540	3.073	3.419	3.672
Index	1.00	1.34	4.81	4.69	5.91	7.15	7.95	8.54
Payments per ton-mile:								
Median payments ..... cents	0.222	0.240	0.218	0.149	0.127	0.123	0.100	0.102
Index	1.00	1.08	0.98	0.67	0.57	0.55	0.45	0.46
PART 2.—TOTAL USER-TAX PAYMENTS AT 1955 RATES IN 11 STATES[1]								
Payments per year:								
Median payments ..... dollars	41	48	182	866	881	1,220	1,860	2,214
Index	1.00	1.17	4.44	21.12	21.49	29.76	45.37	54.00
Payments per mile:								
Median payments ..... cents	0.411	0.600	1.820	1.732	2.318	3.050	3.720	4.428
Index	1.00	1.36	4.13	3.93	5.26	6.92	8.44	10.04
Payments per ton-mile:								
Median payments ..... cents	0.227	0.250	0.192	0.128	0.116	0.122	0.109	0.123
Index	1.00	1.10	0.85	0.56	0.51	0.54	0.48	0.54

[1] The 9 States included in part 1 plus Oregon, which has a tax structure based on the findings of incremental studies, and Idaho, which has a mileage tax similar in coverage to those in Colorado, New York, Ohio, and Oregon.

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[1] The median is the middle value of a series arranged in order of magnitude; for example, in the series of numbers, 27, 36, 40, 51, 63, the median is 40. In the case of an even numbered series the average of the two middle values is taken.

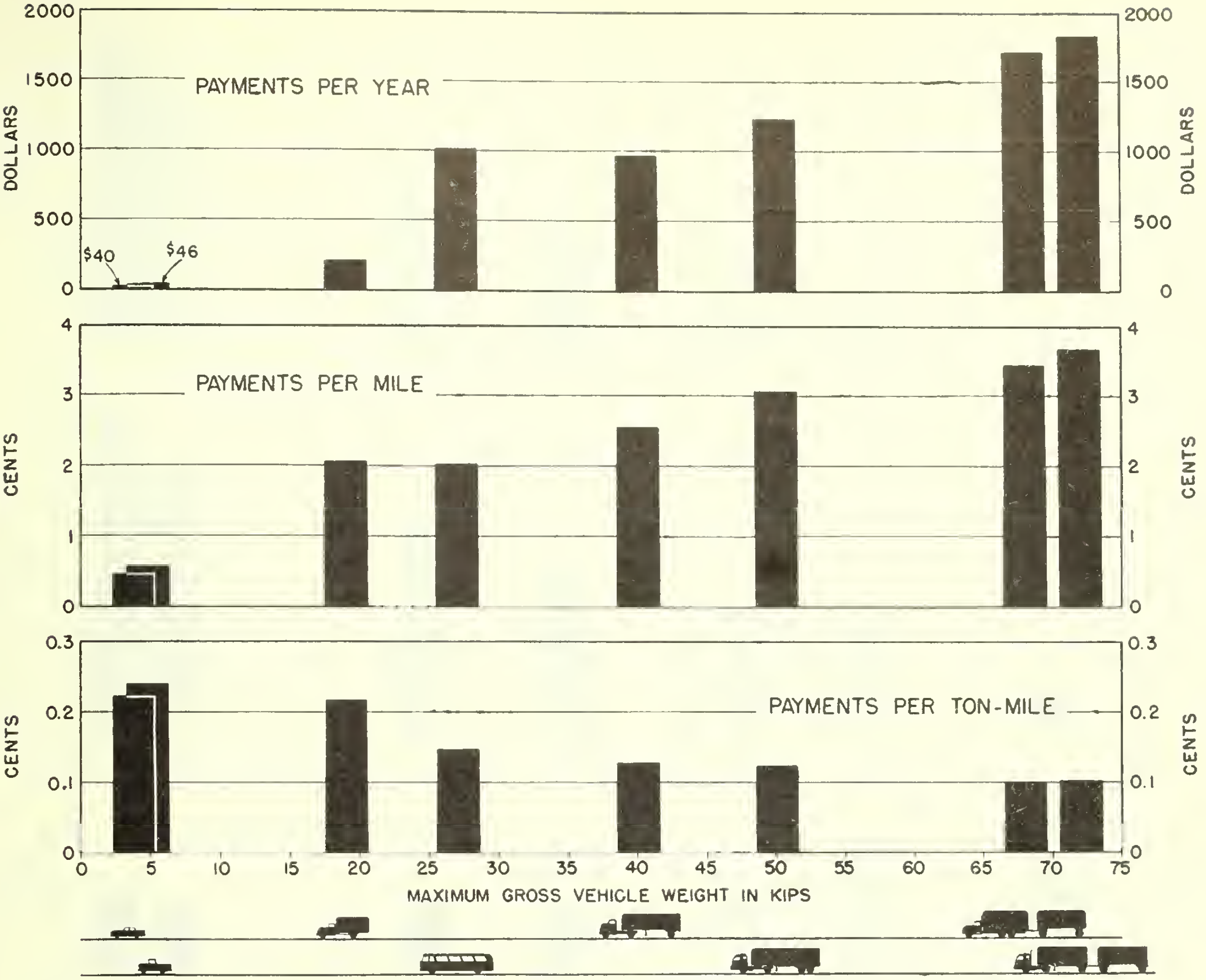

Figure 1.—Median user-tax payments under tax-study recommendations in nine States.

State-by-State Comparisons of Tax-Study Recommendations

The data on tax-study recommendations for the nine States for which comparable data were available are presented in tables 3, 5, and 6, the values being given respectively in terms of tax payments per year, per mile, and per ton-mile. In the third column of each table there is given a series of symbols indicating for each State the type of tax study or studies on which the findings in that State were based. As indicated by the symbol "T" the majority of studies were based on the gross ton-mile concept, previously discussed. Of the two studies made in Ohio the one symbolized "S" was conducted by the standard-cost method which, as applied to motor-vehicle taxation, was a modified ton-mile solution.

The incremental method, denoted by the symbol "I" and used in Louisiana, Minnesota, and Ohio, is based on the concept that successive increments of highway cost are occasioned by vehicle groups of successively

Table 3.—Total user-tax payments per year (in dollars) for selected vehicles under tax-study recommendations

State	Year tax study published	Method[1]	No. 1: Passenger car		Single-unit trucks				No. 4: Bus		Truck combinations							
					No. 2: Pickup		No. 3: 2-axle, 6-tire				Tractor-semitrailer				No. 7: Truck-trailer		No. 8: Tractor-semitrailer-trailer	
											No. 5: 3-axle		No. 6: 4-axle					
			Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index
California	1946	T	25	1.00	29	1.16	128	5.12	870	34.80	885	35.40	1,135	45.40	1,672	66.88	1,847	73.88
Colorado [2]	1950	T	55	1.00	51	.93	292	5.31	1,749	31.80	2,423	44.05	3,112	56.58	5,126	93.20	5,392	98.04
Colorado [3]	1950	T	52	1.00	49	.94	282	5.42	1,665	32.02	2,265	43.56	2,921	56.17	4,841	93.10	5,098	98.04
Illinois	1948	T	40	1.00	51	1.28	264	6.60	972	24.30	1,161	29.03	1,241	31.03	1,479	36.98	1,500	37.50
Louisiana	1955	IC	46	1.00	46	1.00	220	4.78	1,009	21.93	934	20.30	1,217	26.46	1,680	36.52	(4)	-----
Minnesota	1954	I	52	1.00	47	.90	207	3.98	2,063	39.67	1,234	23.73	1,833	35.24	(4)	-----	(4)	-----
New York [5]	1950	T	47	1.00	65	1.38	287	6.11	1,255	26.70	1,540	32.77	2,639	56.15	(4)	-----	(4)	-----
New York [6]	1950	T	22	1.00	26	1.18	106	4.82	549	24.95	610	27.73	1,010	45.91	(4)	-----	(4)	-----
Ohio	1951	S	38	1.00	58	1.53	169	4.45	1,107	29.13	1,016	26.74	1,351	35.55	1,651	43.45	2,310	60.79
Ohio	1953	I	39	1.00	38	.97	208	5.33	1,389	35.62	996	25.54	1,353	34.69	2,327	59.67	2,118	54.31
Utah	1950	T	33	1.00	36	1.09	159	4.82	(7)	-----	836	25.33	1,156	35.03	1,739	52.70	1,825	55.30
Washington	1948	T	40	1.00	36	.90	177	4.43	982	24.55	790	19.75	1,061	26.53	1,874	46.85	1,766	44.15

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[1] Method used to allocate tax responsibility indicated thus: I=Incremental; IC=Recommendations based on findings of incremental and cost-function solutions; S=Standard cost; T=Ton mile.

[2] Motor-fuel tax rate of 8.5 cents per gallon and low registration fees.

[3] Motor-fuel tax rate of 6.5 cents per gallon and high registration fees.

[4] Vehicle combination not permitted by State size and weight regulations.

[5] Maximum expenditure program with motor-fuel tax rate of 4 cents per gallon.

[6] Continuation of "current" (1949) expenditure level with motor-fuel tax rate of 3 cents per gallon.

[7] No recommendation.

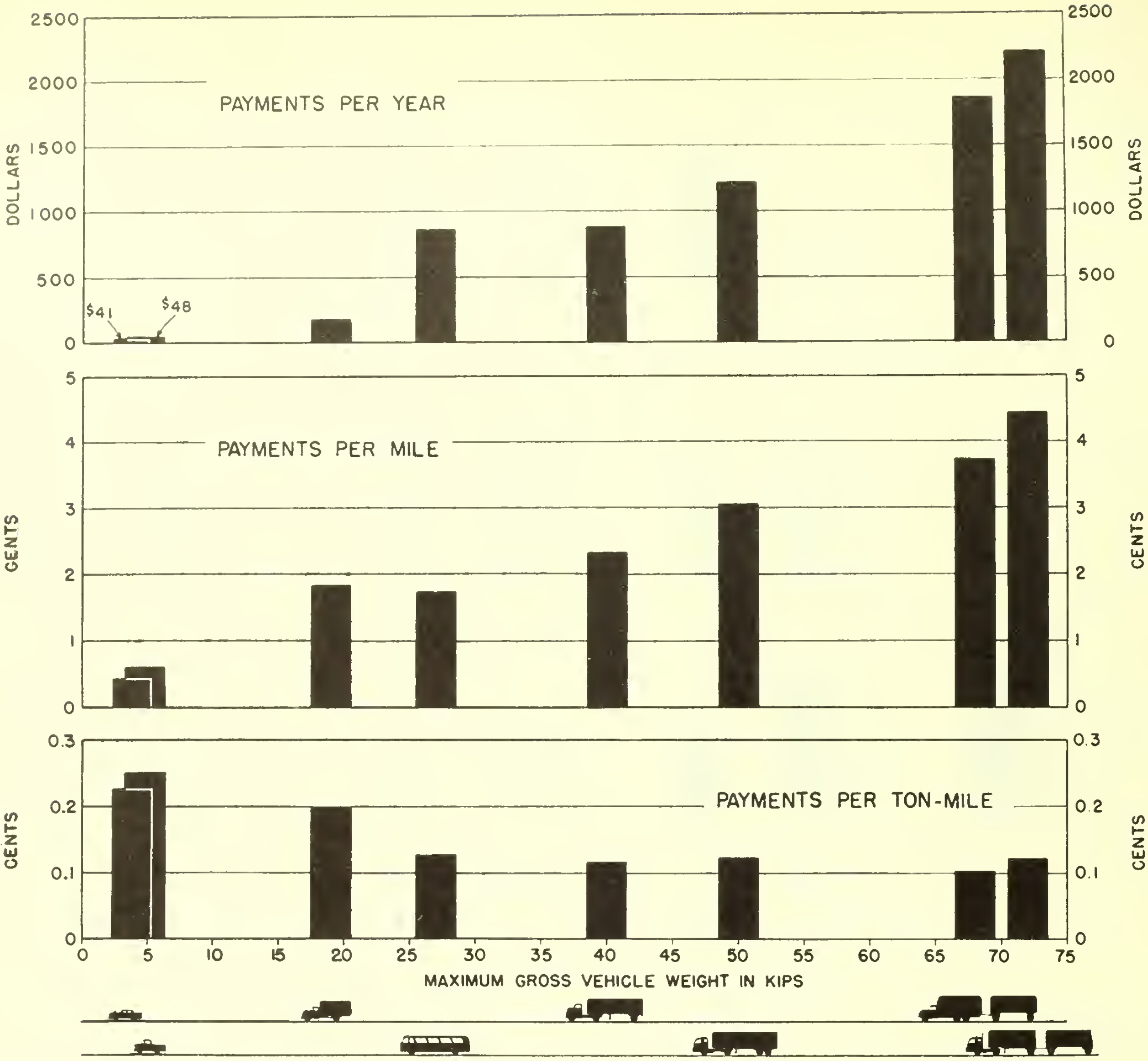

Figure 2.—Median user-tax payments at 1955 rates in 11 States (includes the 9 States for which tax-study recommendations are reported in figure 1 plus Idaho and Oregon).

greater size and weight. The cost-function method, which was combined with an incremental study to produce the Louisiana findings, divides highway costs into three groups: (*1*) Those assignable on a per-vehicle basis, (*2*) those assignable on a per-mile basis, and (*3*) those assignable on a weight or ton-mile basis.

It will be noted that two sets of findings are reported for Colorado and New York, as well as for Ohio. Since the investigators in both Colorado and New York offered several alternatives and expressed no decided preference for any one of them, two proposals were selected to illustrate the findings in each of those States. In the Colorado case two different assumptions were made regarding the relative magnitudes of gasoline-tax rates and registration fees. In New York two different levels of required revenue were postulated.

A glance at each column of table 3 (p. 75) will disclose a wide disparity among the States in the study findings for the several selected vehicles. Similar glances along successive lines will reveal no consistent pattern of upward variation with size of vehicle. Several reasons can be definitely assigned to account for this dispersion. The principal reason, perhaps, lies in the relative magnitudes of the highway programs which the various recommended tax schedules were designed to finance. States differ in the relative extent of their highway needs, and also in the adequacy of the programs designed to meet those needs. The number of motor vehicles in relation to the required revenues affects the general level of user taxation; and the composition of the vehicle population with respect to size and weight may affect the charges to the several size groups.

Table 4.—Comparison of tax-study findings for selected truck combinations indicating the range of required tax payments

Class interval of required tax payment	Number of tax studies		
	No 5: 3-axle tractor-semi-trailer	No 6: 4-axle tractor-semi-trailer	No 8: 5-axle tractor-semi-trailer-trailer
Dollars			
500–749	1	-----	-----
750–999	5	-----	-----
1,000–1,249	3	6	-----
1,250–1,499	-----	2	-----
1,500–1,999	1	1	4
2,000–2,999	2	2	2
3,000 and over	-----	1	2

The study findings are naturally affected by the particular method (incremental, ton-mile, etc.) used for the assignment of tax responsibility. There is a tendency also for investigators, when converting the tax-study results into the form of a recommended schedule of tax rates, to modify, in the interest of simplicity and uniformity, the findings derived from the strict application of theory or formulated procedure.

In spite of the apparent shotgun scatter of study findings, some rudiments of a pattern, or at least a discernible central tendency, begin to emerge when the recommended payments for a given vehicle are grouped by intervals as shown in table 4.

It will be observed that for all three truck combinations the modal class interval (the interval within and about which the findings cluster) lies in the lower part of the total range of values covered by the study findings.

Turning to the lighter weight vehicles, we find that 6 out of 12 tax studies recommended annual passenger-car payments lying within the interval $37.50 to $50. In the case of the pickup truck the central tendency is weak, the recommendations being distributed rather evenly over a range extending from $26 to $65. So also with the bus, for which the findings are widely scattered over a range from $549 to $2,063. By contrast 7 out of 12 tax studies recommend annual payments for the 2-axle, 6-tire truck lying between $200 and $300.

Table 5.—Total user-tax payments per mile (in cents) for selected vehicles under tax-study recommendations

State	Year tax study published	Method[1]	No. 1: Passenger car		Single-unit trucks				No. 4: Bus		Truck combinations							
					No. 2: Pickup		No. 3: 2-axle, 6-tire				Tractor-semitrailer				No. 7: Truck-trailer		No. 8: Tractor-semitrailer-trailer	
											No. 5: 3-axle		No. 6: 4-axle					
			Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index
California	1946	T	0.269	1.00	0.363	1.35	1.280	4.76	1.740	6.47	2.329	8.66	2.838	10.55	3.344	12.43	3.694	13.73
Colorado[2]	1950	T	.591	1.00	.638	1.08	2.920	4.94	3.498	5.92	6.376	10.79	7.780	13.16	10.252	17.35	10.784	18.25
Colorado[3]	1950	T	.559	1.00	.613	1.10	2.820	5.04	3.330	5.96	5.961	10.67	7.303	13.06	9.682	17.32	10.196	18.24
Illinois	1948	T	.430	1.00	.638	1.48	2.640	6.14	1.944	4.52	3.055	7.10	3.103	7.22	2.958	6.88	3.000	6.98
Louisiana	1955	IC	.495	1.00	.575	1.16	2.200	4.44	2.018	4.08	2.458	4.97	3.043	6.15	3.360	6.79	(4)	-----
Minnesota	1954	I	.559	1.00	.588	1.05	2.070	3.70	4.126	7.38	3.247	5.81	4.583	8.20	(4)	-----	(4)	-----
New York[5]	1950	T	.505	1.00	.813	1.61	2.870	5.68	2.510	4.97	4.053	8.03	6.598	13.06	(4)	-----	(4)	-----
New York[6]	1950	T	.237	1.00	.325	1.37	1.060	4.47	1.098	4.63	1.605	6.77	2.525	10.65	(4)	-----	(4)	-----
Ohio	1951	S	.409	1.00	.725	1.77	1.690	4.13	2.214	5.41	2.674	6.54	3.378	8.26	3.302	8.07	4.620	11.30
Ohio	1953	I	.419	1.00	.475	1.13	2.080	4.96	2.778	6.63	2.621	6.26	3.383	8.07	4.654	11.11	4.236	10.11
Utah	1950	T	.355	1.00	.450	1.27	1.590	4.48	(7)	-----	2.200	6.20	2.890	8.14	3.478	9.80	3.650	10.28
Washington	1948	T	.430	1.00	.450	1.05	1.770	4.12	1.964	4.57	2.079	4.83	2.653	6.17	3.748	8.72	3.532	8.21

[1] Method used to allocate tax responsibility indicated thus: I=Incremental; IC=Recommendations based on findings of incremental and cost-function solutions; S=Standard cost; T=Ton mile.

[2] Motor-fuel tax rate of 8.5 cents per gallon and low registration fees.

[3] Motor-fuel tax rate of 6.5 cents per gallon and high registration fees.

[4] Vehicle combination not permitted by State size and weight regulations.

[5] Maximum expenditure program with motor-fuel tax rate of 4 cents per gallon.

[6] Continuation of "current" (1949) expenditure level with motor-fuel tax rate of 3 cents per gallon.

[7] No recommendation.

Table 6.—Total user-tax payments per ton-mile (in cents) for selected vehicles under tax-study recommendations

State	Year tax study published	Method[1]	No. 1: Passenger car		Single-unit trucks				No. 4: Bus		Truck combinations							
					No. 2: Pickup		No. 3: 2-axle, 6-tire				Tractor-semitrailer				No. 7: Truck-trailer		No. 8: Tractor-semitrailer-trailer	
											No. 5: 3-axle		No. 6: 4-axle					
			Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index
California	1946	T	0.139	1.00	0.151	1.09	0.135	0.97	0.129	0.93	0.116	0.83	0.114	0.82	0.098	0.71	0.103	0.74
Colorado[2]	1950	T	.304	1.00	.266	.87	.307	1.01	.259	.85	.319	1.05	.311	1.02	.302	.99	.299	.98
Colorado[3]	1950	T	.288	1.00	.255	.89	.297	1.03	.247	.86	.298	1.03	.292	1.01	.285	.99	.283	.98
Illinois	1948	T	.222	1.00	.266	1.20	.278	1.25	.144	.65	.153	.69	.124	.56	.087	.39	.083	.37
Louisiana	1955	IC	.255	1.00	.240	.94	.232	.91	.149	.58	.123	.48	.122	.48	.099	.39	(4)	-----
Minnesota	1954	I	.288	1.00	.245	.85	.218	.76	.306	1.06	.162	.56	.183	.64	(4)	-----	(4)	-----
New York[5]	1950	T	.260	1.00	.339	1.30	.302	1.16	.186	.72	.203	.78	.264	1.02	(4)	-----	(4)	-----
New York[6]	1950	T	.122	1.00	.135	1.11	.112	.92	.081	.66	.080	.66	.101	.83	(4)	-----	(4)	-----
Ohio	1951	S	.211	1.00	.302	1.43	.178	.84	.164	.78	.134	.64	.135	.64	.097	.46	.128	.61
Ohio	1953	I	.216	1.00	.198	.92	.219	1.01	.206	.95	.131	.61	.135	.62	.137	.63	.118	.55
Utah	1950	T	.183	1.00	.187	1.02	.167	.91	(7)	-----	.110	.60	.116	.63	.102	.56	.101	.55
Washington	1948	T	.222	1.00	.187	.84	.186	.84	.145	.65	.104	.47	.106	.48	.110	.50	.098	.44

[1] Method used to allocate tax responsibility indicated thus: I=Incremental; IC=Recommendations based on findings of incremental and cost-function solutions; S=Standard cost; T=Ton mile.

[2] Motor-fuel tax rate of 8.5 cents per gallon and low registration fees.

Motor-fuel tax rate of 6.5 cents per gallon and high registration fees.

[4] Vehicle combination not permitted by State size and weight regulations.

[5] Maximum expenditure program with motor-fuel tax rate of 4 cents per gallon.

[6] Continuation of "current" (1949) expenditure level with motor-fuel tax rate of 3 cents per gallon.

[7] No recommendation.

Table 7.—Total user-tax payments per year (in dollars) for selected vehicles at 1955 tax rates in selected States

State	No. 1: Passenger car		Single-unit trucks				No. 4: Bus		Truck combinations							
			No. 2: Pickup		No. 3: 2-axle, 6-tire				Tractor-semitrailer				No. 7: Truck-trailer		No. 8: Tractor-semitrailer-trailer	
									No. 5: 3-axle		No. 6: 4-axle					
	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index
California	41	1.00	53	1.29	152	3.71	734	17.90	650	15.85	806	19.66	1,232	30.05	1,230	30.00
Colorado	39	1.00	40	1.03	168	4.31	1,668	42.77	1,211	31.05	1,529	39.21	2,505	64.23	2,627	67.36
Idaho	51	1.00	48	.94	191	3.75	979	19.20	1,082	21.22	1,415	27.75	2,359	46.25	2,491	48.84
Illinois	38	1.00	46	1.21	223	5.87	866	22.79	1,036	27.26	1,265	33.29	1,860	48.95	1,937	50.97
Louisiana	42	1.00	46	1.10	190	4.52	917	21.83	834	19.86	1,007	23.98	1,440	34.29	(1)	-----
Minnesota	51	1.00	51	1.00	154	3.02	1,748	34.27	756	14.82	986	19.33	(1)	-----	(1)	-----
New York	37	1.00	46	1.24	182	4.92	467	12.62	881	23.81	1,220	32.97	(1)	-----	(1)	-----
Ohio	38	1.00	62	1.63	208	5.47	865	22.76	1,057	27.82	1,467	38.61	2,632	69.26	2,542	66.89
Oregon	43	1.00	53	1.23	216	5.02	1,187	27.60	1,304	30.33	1,765	41.05	3,050	70.93	3,225	75.00
Utah	33	1.00	33	1.00	124	3.76	812	24.61	546	16.55	691	20.94	1,219	36.94	1,165	35.30
Washington	41	1.00	48	1.17	158	3.85	743	18.12	675	16.46	944	23.02	1,594	38.88	1,446	35.27

[1] Vehicle combination not permitted by State size and weight regulations.

Table 8.—Comparison of tax payments per year required of a 50,000-pound, 4-axle tractor-semitrailer (No. 6) under tax-study recommendations and under 1955 rates in nine States

State	Tax payments per year: Tax-study recommendations	Tax payments per year: Required tax payments at 1955 rates	Excess of tax-study recommendations over 1955 payments
California	$1,135	$806	$329
Colorado [1]	3,112	1,529	1,583
Colorado [2]	2,921		1,392
Illinois	1,241	1,265	−24
Louisiana	1,217	1,007	210
Minnesota	1,833	986	847
New York [3]	2,639	1,220	1,419
New York [4]	1,010		−210
Ohio [5]	1,351	1,467	−116
Ohio [6]	1,353		−114
Utah	1,156	691	465
Washington	1,061	944	117

[1] Motor-fuel tax rate of 8.5 cents per gallon and low registration fees.
[2] Motor-fuel tax rate of 6.5 cents per gallon and high registration fees.
[3] Maximum expenditure program with motor-fuel tax rate of 4 cents per gallon.
[4] Continuation of "current" (1949) expenditure level with motor-fuel tax rate of 3 cents per gallon.
[5] Standard-cost method used to allocate tax responsibility.
[6] Incremental method used to allocate tax responsibility.

In table 5 (p. 77) the study findings are expressed in required user-tax payments per mile. The range of variation among the several vehicle classes is narrowed by the elimination of the mileage factor. From State to State the pattern is unchanged by this conversion, and comparisons would reveal the same central tendency.

Recommended payments per ton-mile are shown in table 6 (p. 77). The ton-miles used in computing this table are the products of assumed annual mileage and maximum gross weight. Had average operating weights been used instead of maximum gross weights, the payments for the heavier vehicles would have been higher and the indexes relative to the passenger-car value would generally have been nearer to unity.

There is a general, although by no means entirely consistent, tendency for the recommended payments per ton-mile to decline as the size of vehicle increases. This trend is evidenced even in those States where a gross ton-mile solution was used. An exception occurs in the case of Colorado, where the values hover about 3 mills per ton-mile throughout the range of vehicle size.

Although size and weight as well as miles traveled are important considerations in motor-vehicle taxation, there is no scientific basis for the contention that such taxes should be made directly proportional to gross ton-miles. For example, the required thickness of a slab or beam varies as the square root of the load, rather than directly with the load itself. Numerous students of the subject have rejected the ton-mile theory and have resorted to the incremental solution, which attempts to assign tax responsibility in proportion to the costs occasioned by the traffic of vehicles of different sizes.

State-by-State Comparisons of Tax Payments at 1955 Rates

Tables 7 (p. 77), 9, and 10 give corresponding values of actual required tax payments at 1955 rates, per year, per mile, and per ton-mile, respectively. Values for Idaho and Oregon are given in addition to those for the nine tax-study States.

The appearance and general import of these tables are not unlike what is found in the tax-study tables, 3, 5, and 6. It is of interest nonetheless to compare the actual required tax payments in 1955 with the recommendations of the several tax studies. A single example, that of the 50,000-pound, 4-axle tractor-semitrailer—a widely used type of combination—will suffice. The comparison is given in table 8 for the required tax payments per year.

Quite evidently the general trend is for the tax laws enacted by the State legislatures to fall short of the tax-study recommendations. In 8 of the 12 comparisons shown the required payments are less than those recommended. In Illinois and Ohio the required payments somewhat exceed the tax-study recommen-

Table 9.—Total user-tax payments per mile (in cents) for selected vehicles at 1955 tax rates in selected States

State	No. 1: Passenger car		Single-unit trucks				No. 4: Bus		Truck combinations							
			No. 2: Pickup		No. 3: 2-axle, 6-tire				Tractor-semitrailer				No. 7: Truck-trailer		No. 8: Tractor-semitrailer-trailer	
									No. 5: 3-axle		No. 6: 4-axle					
	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index
California	0.441	1.00	0.663	1.50	1.520	3.45	1.468	3.33	1.711	3.88	2.015	4.57	2.464	5.59	2.460	5.58
Colorado	.419	1.00	.500	1.19	1.680	4.01	3.336	7.96	3.187	7.61	3.823	9.12	5.010	11.96	5.254	12.54
Idaho	.548	1.00	.600	1.09	1.910	3.49	1.958	3.57	2.847	5.20	3.538	6.46	4.718	8.61	4.982	9.09
Illinois	.409	1.00	.575	1.41	2.230	5.45	1.732	4.23	2.726	6.66	3.163	7.73	3.720	9.10	3.874	9.47
Louisiana	.452	1.00	.575	1.27	1.900	4.20	1.834	4.06	2.195	4.86	2.518	5.57	2.880	6.37	(1)	---
Minnesota	.548	1.00	.638	1.16	1.540	2.81	3.496	6.38	1.989	3.63	2.465	4.50	(1)	-----	(1)	---
New York	.398	1.00	.575	1.44	1.820	4.57	.934	2.35	2.318	5.82	3.050	7.66	(1)	-----	(1)	---
Ohio	.409	1.00	.775	1.89	2.080	5.09	1.730	4.23	2.782	6.80	3.668	8.97	5.264	12.87	5.084	12.43
Oregon	.462	1.00	.663	1.44	2.160	4.68	2.374	5.14	3.432	7.43	4.413	9.55	6.100	13.20	6.450	13.96
Utah	.355	1.00	.413	1.16	1.240	3.49	1.624	4.57	1.437	4.05	1.728	4.87	2.438	6.87	2.330	6.56
Washington	.441	1.00	.600	1.36	1.580	3.58	1.486	3.37	1.776	4.03	2.360	5.35	3.188	7.23	2.892	6.56

[1] Vehicle combination not permitted by State size and weight regulations.

Table 10.—Total user-tax payments per ton-mile (in cents) for selected vehicles at 1955 tax rates in selected States

State	No. 1: Passenger car		Single-unit trucks				No. 4: Bus		Truck combinations							
			No. 2: Pickup		No. 3: 2-axle, 6-tire				Tractor-semitrailer				No. 7: Truck-trailer		No. 8: Tractor-semitrailer-trailer	
									No. 5: 3-axle		No. 6: 4-axle					
	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index	Payment	Index
California	0.227	1.00	0.276	1.22	0.160	0.70	0.109	0.48	0.086	0.38	0.081	0.36	0.072	0.32	0.068	0.30
Colorado	.216	1.00	.208	.96	.177	.82	.247	1.14	.159	.74	.153	.71	.147	.68	.146	.68
Idaho	.282	1.00	.250	.89	.201	.71	.145	.51	.142	.50	.142	.50	.139	.49	.138	.49
Illinois	.211	1.00	.240	1.14	.235	1.11	.128	.61	.136	.64	.127	.60	.109	.52	.108	.51
Louisiana	.233	1.00	.240	1.03	.200	.86	.136	.58	.110	.47	.101	.43	.085	.36	(1)	-----
Minnesota	.282	1.00	.266	.94	.162	.57	.253	.90	.099	.35	.099	.35	(1)	-----	(1)	-----
New York	.205	1.00	.240	1.17	.192	.94	.069	.34	.116	.57	.122	.60	(1)	-----	(1)	-----
Ohio	.211	1.00	.323	1.53	.219	1.04	.128	.61	.139	.66	.147	.70	.155	.73	.141	.67
Oregon	.238	1.00	.276	1.16	.227	.95	.176	.74	.172	.72	.177	.74	.179	.75	.179	.75
Utah	.183	1.00	.172	.94	.131	.72	.120	.66	.072	.39	.069	.38	.072	.39	.065	.36
Washington	.227	1.00	.250	1.10	.166	.73	.110	.48	.089	.39	.094	.41	.094	.41	.080	.35

[1] Vehicle combination not permitted by State size and weight regulations.

dations. In New York the tax-study findings based on continuation of the current expenditure level are lower than the required tax payments at 1955 rates; but the findings based on highway needs (the "maximum expenditure program") are much greater than the actual required payments. The Louisiana report was released only recently and there has been insufficient time for the legislature to deal with its recommendations. In Minnesota no legislative action had been taken subsequent to the tax-study findings.

State-by-State comparisons of required tax payments per mile are set forth in table 9. The comparisons on a ton-mile basis are given in table 10. The downward trend in payments per ton-mile is notable. There is, however, a tendency, in States where weight-distance or similar taxes have been enacted, for the rates per ton-mile on truck combinations to hold steady, or even to increase slightly with increasing size. This trend may be observed in the amounts for Colorado, Idaho, New York, Ohio, and Oregon.

Summary

The results of this brief review of State tax-study recommendations regarding the rates of user-tax payments by vehicles of different sizes and weights are summarized in the following numbered paragraphs:

1. Median values of tax-study findings range from $40 per year for a light passenger car to $1,836 per year for a 5-axle tractor semitrailer-trailer combination; and from 0.43 to 3.67 cents per mile of travel.

2. With respect to the rate of recommended tax payments per gross ton-mile, the trend of median tax-study values is downward. The variation among the vehicles selected for study extends from 2.2 mills per ton-mile for the light passenger car to 1.2 mills for the 4-axle tractor-semitrailer and 1.0 mill for the 5-axle tractor-semitrailer-trailer.

3. There is a wide variation from State to State in the recommended user-tax payments for each of the eight vehicles selected for comparison. In spite of this dispersion there is a marked central tendency, particularly among the heavy vehicle combinations, in that the study findings tend to group within relatively narrow intervals. Thus, for the 3-axle tractor-semitrailer, 5 out of 12 studies recommend annual tax payments within the range $750 to $1,000. In 6 out of 12 studies the recommended payment for the 4-axle tractor-semitrailer lies between $1,000 and $1,250 per year. For the largest combination in the group, the 5-axle tractor-semitrailer-trailer, 4 out of 8 studies recommended annual tax payments lying between $1,500 and $2,000.

4. Comparison of the tax-study findings with the payments required under 1955 tax rates reveals a general tendency (with exceptions) for the tax laws enacted by State legislatures to fall short of tax-study recommendations in the payments required of heavy motor vehicles and combinations. For example, 8 out of 12 tax-study findings recommended greater user-tax payments for the 4-axle tractor-semitrailer than were imposed under the corresponding 1955 tax laws in the nine States in which the studies were made.

Appendix A

Tables 11–18 give the results of this analysis in full. Each table presents the data for one of the eight selected vehicles, beginning with the passenger car in table 11 and ending with the 5-axle tractor-semitrailer-trailer combination in table 18.

Part 1 of each table gives, for the particular vehicle, the recommended tax payments as derived from the tax studies in all of the nine States. Part 1 is again divided into two sections. That on the left gives the tax-study findings according to the values of annual mileage and miles per gallon used for the given vehicle in each study. In the case of those States where registered gross weight was used as a basis of vehicle grouping, the required tax payments were evaluated directly for the given vehicle group. In States where a different vehicle grouping was used (Ohio, for example, registers vehicles by empty weight), it was necessary to determine into what group according to the State's classification the vehicle, as described in table 1, would properly fall. The information obtained and tabulated in this manner was omitted from the main body of the report because of the defectiveness of State-to-State comparisons.

The right-hand section of part 1 in tables 11–18 gives the tax-study findings as computed on a uniform basis by the use of the values of gross weight, annual mileage, and miles per gallon listed in table 1. Part 2 gives the user-tax payments as required under tax rates prevailing in 1955. In all three sections of these tables values are given in terms of required tax payment per year, per mile, and per ton-mile.

The most pertinent comparisons among vehicles and among States are discussed in the text. The inclusion of tables 11–18 in the appendix provides the opportunity for more detailed comparisons, with particular reference to the tax-study findings on the bases established in the individual studies made in nine States.

Table 11.—Tax payments under tax-study recommendations and at 1955 tax rates in selected States on a lightweight passenger car (No. 1)

State	Part 1.—Tax study recommendations											Part 2.—Total 1955 user-tax payments based on uniform amount of travel and fuel usage [2]		
	Year tax study published	Method [1]	State's own basis					Total user-tax payments based on uniform amount of travel and fuel usage [2]					
			Annual travel	Fuel consumption rate	Total user-tax payments								
					Per year	Per mile	Per ton-mile	Per year	Per mile	Per ton-mile	Per year	Per mile	Per ton-mile
			Miles	*Miles/gal.*	*Dollars*	*Cents*	*Cents*	*Dollars*	*Cents*	*Cents*	*Dollars*	*Cents*	*Cents*
California	1946	T	8,554	15.5	25	0.292	0.150	25	0.269	0.139	41	0.441	0.227
Colorado [3]	1950	T	9,200	16.3	56	.609	.314	55	.591	.304	39	.419	.216
Colorado [4]	1950	T	9,200	16.3	53	.576	.297	52	.559	.288	-----	-----	-------
Idaho	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	51	.548	.282
Illinois	1948	T	9,250	15.5	42	.454	.234	40	.430	.222	38	.409	.211
Louisiana	1955	IC	10,143	14.8	55	.542	.279	46	.495	.255	42	.452	.233
Minnesota	1954	I	8,935	14.9	55	.616	.317	52	.559	.288	51	.548	.282
New York [5]	1950	T	8,500	15.7	47	.553	.285	47	.505	.260	37	.398	.205
New York [6]	1950	T	8,500	15.7	21	.247	.127	22	.237	.122	-----	-----	-------
Ohio	1951	S	9,230	15.9	39	.423	.218	38	.409	.211	38	.409	.211
Ohio	1953	I	9,235	15.0	42	.455	.234	39	.419	.216	-----	-----	-------
Oregon	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	43	.462	.238
Utah	1950	T	10,000	15.5	37	.370	.191	33	.355	.183	33	.355	.183
Washington	1948	T	8,085	16.2	36	.445	.229	40	.430	.222	41	.441	.227

[1] Method used to allocate tax responsibility indicated thus: I=Incremental; IC=Recommendations based on findings of incremeutal and cost-function solutions; S=Standard cost; T=Ton-mile.

[2] Travel, 9,300 miles per year; fuel consumption rate, 16.7 miles per gallon.

[3] Motor-fuel tax rate of 8.5 cents per gallon and low registration fees.

[4] Motor-fuel tax rate of 6.5 cents per gallon and high registration fees.

[5] Maximum expenditure program with fuel-tax rate of 4 cents per gallon.

[6] Continuation of "current" (1949) expenditure level with fuel-tax rate of 3 cents per gallon.

Table 12.—Tax payments under tax-study recommendations and at 1955 tax rates in selected States on a pickup truck (No. 2)

State	Part 1.—Tax study recommendations										Part 2.—Total 1955 user-tax payments based on uniform amount of travel and fuel usage [3]		
	Year tax study published	Method [1]	State's own basis					Total user-tax payments based on uniform amount of travel and fuel usage [2]					
			Annual travel	Fuel consumption rate	Total user-tax payments								
					Per year	Per mile	Per ton-mile	Per year	Per mile	Per ton-mile	Per year	Per mile	Per ton-mile
			Miles	*Miles/gal.*	*Dollars*	*Cents*	*Cents*	*Dollars*	*Cents*	*Cents*	*Dollars*	*Cents*	*Cents*
California	1946	T	11,400	15.5	37	0.325	0.135	29	0.363	0.151	53	0.663	0.276
Colorado [3]	1950	T	8,500	14.5	57	.671	.280	51	.638	.266	40	.500	.208
Colorado [4]	1950	T	8,500	14.5	54	.635	.265	49	.613	.255	-----	-----	-------
Idaho	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	48	.600	.250
Illinois	1948	T	10,000	14.3	60	.600	.250	51	.638	.266	46	.575	.240
Louisiana	1955	IC	7,305	13.0	49	.671	.280	46	.575	.240	46	.575	.240
Minnesota	1954	I	5,064	13.3	38	.750	.312	47	.588	.245	51	.638	.266
New York [5]	1950	T	8,500	12.8	71	.835	.348	65	.813	.339	46	.575	.240
New York [6]	1950	T	8,500	12.8	30	.353	.147	26	.325	.135	-----	-----	-------
Ohio	1951	S	12,214	12.5	81	.663	.276	58	.725	.302	62	.775	.323
Ohio	1953	I	10,700	13.0	53	.495	.206	38	.475	.198	-----	-----	-------
Oregon	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	53	.663	.276
Utah	1950	T	8,600	15.17	38	.442	.184	36	.450	.187	33	.413	.172
Washington	1948	T	8,068	12.75	44	.545	.227	36	.450	.187	48	.600	.250

[1] Method used to allocate tax responsibility indicated thus: I=Incremental; IC=Recommendations based on findings of incremental and cost-function solutions; S=Standard cost; T=Ton-mile.

[2] Travel, 8,000 miles per year; fuel consumption rate, 15.6 miles per gallon.

[3] Motor-fuel tax rate of 8.5 cents per gallon and low registration fees.

[4] Motor-fuel tax rate of 6.5 cents per gallon and high registration fees.

[5] Maximum expenditure program with fuel-tax rate of 4 cents per gallon.

[6] Continuation of "current" (1949) expenditure level with fuel-tax rate of 3 cents per gallon.

Table 13.—Tax payments under tax-study recommendations and at 1955 tax rates in selected States on a 2-axle, 6-tire truck (No. 3)

State	Part 1.—Tax study recommendations										Part 2.—Total 1955 user-tax payments based on uniform amount of travel and fuel usage [2]		
	Year tax study published	Method [1]	State's own basis					Total user-tax payments based on uniform amount of travel and fuel usage [2]					
			Annual travel	Fuel consumption rate	Total user-tax payments								
					Per year	Per mile	Per ton-mile	Per year	Per mile	Per ton-mile	Per year	Per mile	Per ton-mile
			Miles	*Miles/gal.*	*Dollars*	*Cents*	*Cents*	*Dollars*	*Cents*	*Cents*	*Dollars*	*Cents*	*Cents*
California	1946	T	17,500	8.83	212	1.211	0.127	128	1.280	0.135	152	1.520	0.160
Colorado [3]	1950	T	13,900	7.4	343	2.468	.260	292	2.920	.307	168	1.680	.177
Colorado [4]	1950	T	13,900	7.4	321	2.309	.243	282	2.820	.297	-----	-----	-------
Idaho	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	191	1.910	.201
Illinois	1948	T	23,500	7.8	351	1.494	.157	264	2.640	.278	223	2.230	.235
Louisiana	1955	IC	25,344	5.8	436	1.723	.181	220	2.200	.232	190	1.900	.200
Minnesota	1954	I	15,000	8.1	247	1.647	.173	207	2.070	.218	154	1.540	.162
New York [5]	1950	T	15,000	8.0	311	2.073	.218	287	2.870	.302	182	1.820	.192
New York [6]	1950	T	15,000	8.0	123	.820	.086	106	1.060	.112	-----	-----	-------
Ohio	1951	S	17,593	7.6	221	1.256	.132	169	1.690	.178	208	2.080	.219
Ohio	1953	I	19,800	7.3	279	1.409	.148	208	2.080	.219	-----	-----	-------
Oregon	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	216	2.160	.227
Utah	1950	T	14,300	8.88	176	1.231	.130	159	1.590	.167	124	1.240	.131
Washington	1948	T	11,483	7.0	201	1.750	.184	177	1.770	.186	158	1.580	.166

[1] Method used to allocate tax responsibility indicated thus: I=Incremental; IC=Recommendations based on findings of incremental and cost-function solutions; S=Standard cost; T=Ton-mile.

[2] Travel, 10,000 miles per year; fuel consumption rate, 7.8 miles per gallon.

[3] Motor-fuel tax rate of 8.5 cents per gallon and low registration fees.

[4] Motor-fuel tax rate of 6.5 cents per gallon and high registration fees.

[5] Maximum expenditure program with fuel-tax rate of 4 cents per gallon.

[6] Continuation of "current" (1949) expenditure level with fuel-tax rate of 3 cents per gallon.

Table 14.—Tax payments under tax-study recommendations and at 1955 tax rates in selected States on a 41-passenger bus (No. 4)

State	Part 1.—Tax study recommendations										Part 2.—Total 1955 user-tax payments based on uniform amount of travel and fuel usage [2]		
	Year tax study published	Method [1]	State's own basis					Total user-tax payments based on uniform amount of travel and fuel usage [2]					
			Annual travel	Fuel consumption rate	Total user-tax payments								
					Per year	Per mile	Per ton-mile	Per year	Per mile	Per ton-mile	Per year	Per mile	Per ton-mile
			Miles	*Miles/gal.*	*Dollars*	*Cents*	*Cents*	*Dollars*	*Cents*	*Cents*	*Dollars*	*Cents*	*Cents*
California	1946	T	39,300	4.5	737	1.875	0.139	870	1.740	0.129	734	1.468	0.109
Colorado [3]	1950	T	50,000	5.1	1,780	3.560	.264	1,749	3.498	.259	1,668	3.336	.247
Colorado [4]	1950	T	50,000	5.1	1,689	3.378	.250	1,665	3.330	.247	-----	-----	-----
Idaho	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	979	1.958	.145
Illinois	1948	T	30,000	4.8	813	2.710	.201	972	1.944	.144	866	1.732	.128
Louisiana	1955	IC	65,000	4.8	1,303	2.005	.149	1,009	2.018	.149	917	1.834	.136
Minnesota	1954	I	100,000	6.1	3,944	3.944	.292	2,063	4.126	.306	1,748	3.496	.253
New York [5]	1950	T	27,500	5.2	1,089	3.960	.293	1,255	2.510	.186	467	.934	.069
New York [5]	1950	T	27,500	5.2	424	1.542	.114	549	1.098	.081	-----	-----	-----
Ohio	1951	S	50,000	5.0	1,136	2.272	.168	1,107	2.214	.164	865	1.730	.128
Ohio	1953	I	57,000	7.0	1,323	2.321	.172	1,389	2.778	.206	-----	-----	-----
Oregon	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	1,187	2.374	.176
Utah [7]	1950	T	-----	-----	-----	-----	-----	-----	-----	-----	812	1.624	.120
Washington	1948	T	48,182	4.5	1,065	2.210	.164	982	1.964	.145	743	1.486	.110

[1] Method used to allocate tax responsibility indicated thus: I=Incremental; IC=Recommendations based on findings of incremental and cost-function solutions; S=Standard cost; T=Ton-mile.

[2] Travel, 50,000 miles per year; fuel consumption rate, 5.3 miles per gallon.

[3] Motor-fuel tax rate of 8.5 cents per gallon and low registration fees.

[4] Motor-fuel tax rate of 6.5 cents per gallon and high registration fees.

[5] Maximum expenditure program with fuel-tax rate of 4 cents per gallon.

[6] Continuation of "current" (1949) expenditure level with fuel-tax rate of 3 cents per gallou.

[7] No recommendation.

Table 15.—Tax payments under tax-study recommendations and at 1955 tax rates in selected States on a 3-axle tractor-semitrailer (No. 5)

State	Part 1.—Tax study recommendations										Part 2.—Total 1955 user-tax payments based on uniform amount of travel and fuel usage [2]		
	Year tax study published	Method [1]	State's own basis					Total user-tax payments based on uniform amount of travel and fuel usage [2]					
			Annual travel	Fuel consumption rate	Total user-tax payments								
					Per year	Per mile	Per ton-mile	Per year	Per mile	Per ton-mile	Per year	Per mile	Per ton-mile
			Miles	*Miles/gal.*	*Dollars*	*Cents*	*Cents*	*Dollars*	*Cents*	*Cents*	*Dollars*	*Cents*	*Cents*
California	1946	T	17,500	4.41	424	2.423	0.121	885	2.329	0.116	650	1.711	0.086
Colorado [3]	1950	T	38,000	5.1	2,383	6.271	.314	2,423	6.376	.319	1,211	3.187	.159
Colorado [4]	1950	T	38,000	5.1	2,234	5.879	.294	2,265	5.961	.298	-----	-----	-----
Idaho	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	1,082	2.847	.142
Illinois	1948	T	36,500	4.3	1,189	3.258	.163	1,161	3.055	.153	1,036	2.726	.136
Louisiana	1955	IC	32,500	3.7	1,001	3.080	.154	934	2.458	.123	834	2.195	.110
Minnesota	1954	I	47,002	4.9	1,447	3.079	.154	1,234	3.247	.162	756	1.989	.099
New York [5]	1950	T	30,500	4.3	1,507	4.941	.247	1,540	4.053	.203	881	2.318	.116
New York [6]	1950	T	30,500	4.3	586	1.921	.096	610	1.605	.080	-----	-----	-----
Ohio	1951	S	44,500	4.0	1,181	2.654	.133	1,016	2.674	.134	1,057	2.782	.139
Ohio	1953	I	41,500	4.0	1,114	2.684	.134	996	2.621	.131	-----	-----	-----
Oregon	-----	-----	-----	-----	-----	-----	-----	-----	-----	-----	1,304	3.432	.172
Utah	1950	T	34,100	4.81	794	2.328	.116	836	2.200	.110	546	1.437	.072
Washington	1948	T	25,035	4.4	647	2.584	.129	790	2.079	.104	675	1.776	.089

[1] Method used to allocate tax responsibility indicated thus: I=Incremental; IC=Recommendations based on findings of incremental and cost-function solutions; S=Standard cost; T=Ton mile.

[2] Travel, 38,000 miles per year; fuel consumption rate, 4.8 miles per gallon.

[3] Motor-fuel tax rate of 8.5 cents per gallon and low registration fees.

[4] Motor-fuel tax rate of 6.5 cents per gallon and high registration fees.

[5] Maximum expenditure program with fuel-tax rate of 4 cents per gallon.

[6] Continuation of "current" (1949) expenditure level with fuel-tax rate of 3 cents per gallon.

Table 16.—Tax payments under tax-study recommendations and at 1955 tax rates in selected States on a 4-axle tractor-semitrailer (No. 6)

State	Part 1.—Tax study recommendations										Part 2.—Total 1955 user-tax payments based on uniform amount of travel and fuel usage [2]		
			State's own basis					Total user-tax payments based on uniform amount of travel and fuel usage [2]					
					Total user-tax payments								
	Year tax study published	Method [1]	Annual travel	Fuel consumption rate	Per year	Per mile	Per ton-mile	Per year	Per mile	Per ton-mile	Per year	Per mile	Per ton-mile
			Miles	*Miles/gal.*	*Dollars*	*Cents*	*Cents*	*Dollars*	*Cents*	*Cents*	*Dollars*	*Cents*	*Cents*
California	1946	T	23, 500	3. 86	689	2. 932	0. 117	1, 135	2. 838	0. 114	806	2. 015	0. 081
Colorado [3]	1950	T	40, 000	5. 1	2, 969	7. 423	. 297	3, 112	7. 780	. 311	1, 529	3. 823	. 153
Colorado [4]	1950	T	40, 000	5. 1	2, 812	7. 030	. 281	2, 921	7. 303	. 292	-----	-----	-------
Idaho	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	1, 415	3. 538	. 142
Illinois	1948	T	36, 500	4. 3	1, 189	3, 258	. 130	1, 241	3. 103	. 124	1, 265	3. 163	. 127
Louisiana	1955	IC	32, 500	3. 3	1, 233	3. 794	. 152	1, 217	3. 043	. 122	1, 007	2. 518	. 101
Minnesota	1954	I	47, 002	4. 4	2, 118	4. 506	. 180	1, 833	4. 583	. 183	986	2. 465	. 099
New York [5]	1950	T	36, 500	3. 5	2, 675	7. 329	. 293	2, 639	6. 598	. 264	1, 220	3. 050	. 122
New York [6]	1950	T	36, 500	3. 5	1, 037	2. 841	. 114	1, 010	2. 525	. 101	-----	-----	-------
Ohio	1951	S	46, 700	3. 4	1, 553	3. 325	. 133	1, 351	3. 378	. 135	1, 467	3. 668	. 147
Ohio	1953	I	46, 300	3. 8	1, 484	3. 205	. 128	1, 353	3. 383	. 135	-----	-----	-------
Oregon	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	1, 765	4. 413	. 177
Utah	1950	T	39, 600	4. 57	1, 113	2. 811	. 112	1, 156	2. 890	. 116	691	1. 728	. 069
Washington	1948	T	19, 805	4. 0	762	3. 848	. 154	1, 061	2. 653	. 106	944	2. 360	. 094

[1] Method used to allocate tax responsibility indicated thus: I=Incremental; IC=Recommendations based on findings of incremental and cost-function solutions; S=Standard cost; T=Ton mile.

[2] Travel, 40,000 miles per year; fuel consumption rate, 4.2 miles per gallon.

[3] Motor-fuel tax rate of 8.5 cents per gallon and low registration fees.

[4] Motor-fuel tax rate of 6.5 cents per gallon and high registration fees.

[5] Maximum expenditure program with fuel-tax rate of 4 cents per gallon.

[6] Continuation of "current" (1949) expenditure level with fuel-tax rate of 3 cents per gallon.

Table 17.—Tax payments under tax-study recommendations and at 1955 tax rates in selected States on a truck-trailer combination (No. 7)

State	Part 1.—Tax study recommendations										Part 2.—Total 1955 user-tax payments based on uniform amount of travel and fuel usage [2]		
			State's own basis					Total user-tax payments based on uniform amount of travel and fuel usage [2]					
					Total user-tax payments								
	Year tax study published	Method [1]	Annual travel	Fuel consumption rate	Per year	Per mile	Per ton-mile	Per year	Per mile	Per ton-mile	Per year	Per mile	Per ton-mile
			Miles	*Miles/gal.*	*Dollars*	*Cents*	*Cents*	*Dollars*	*Cents*	*Cents*	*Dollars*	*Cents*	*Cents*
California	1946	T	39, 300	2. 98	1, 394	3. 547	0. 104	1, 672	3. 344	0. 098	1, 232	2. 464	0. 072
Colorado [3]	1950	T	50, 000	5. 1	4, 745	9. 490	. 279	5, 126	10. 252	. 302	2, 505	5. 010	. 147
Colorado [4]	1950	T	50, 000	5. 1	4, 549	9. 098	. 268	4, 841	9. 682	. 285	-----	-----	-------
Idaho	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	2, 359	4. 718	. 139
Illinois	1948	T	36, 500	4. 3	1, 189	3. 258	. 096	1, 479	2. 958	. 087	1, 860	3. 720	. 109
Louisiana	1955	IC	37, 000	2. 7	1, 626	4. 395	. 129	1, 680	3. 360	. 099	1, 440	2. 880	. 085
Minnesota [5]	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	-----	-----	-------
New York [5]	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	-----	-----	-------
Ohio	1951	S	19, 262	2. 5	1, 314	6. 822	. 201	1, 651	3. 302	. 097	2, 632	5. 264	. 155
Ohio	1953	I	55, 500	2. 8	2, 597	4. 679	. 138	2, 327	4. 654	. 137	-----	-----	-------
Oregon	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	3, 050	6. 100	. 179
Utah	1950	T	42, 000	4. 46	1, 496	3. 562	. 105	1, 739	3. 478	. 102	1, 219	2. 438	. 072
Washington	1948	T	26, 333	3. 2	1, 485	5. 639	. 166	1, 874	3. 748	. 110	1, 594	3. 188	. 094

[1] Method used to allocate tax responsibility indicated thus: I=Incremental; IC=Recommendations based on findings of incremental and cost-function solutions; S=Standard cost; T=Ton-mile.

[2] Travel, 50,000 miles per year; fuel consumption rate, 3.5 miles per gallon.

[3] Motor-fuel tax rate of 8.5 cents per gallon and low registration fees.

[4] Motor-fuel tax rate of 6. 5 cents per gallon and high registration fees.

[5] This vehicle combination not permitted by State size and weight regulations.

Table 18.—Tax payments under tax-study recommendations and at 1955 tax rates in selected States on a tractor-semitrailer-trailer combination (No. 8)

State	Part 1.—Tax study recommendations										Part 2.—Total 1955 user-tax payments based on uniform amount of travel and fuel usage [2]		
			State's own basis					Total user-tax payments based on uniform amount of travel and fuel usage [2]					
					Total user-tax payments								
	Year tax study published	Method [1]	Annual travel	Fuel consumption rate	Per year	Per mile	Per ton-mile	Per year	Per mile	Per ton-mile	Per year	Per mile	Per ton-mile
			Miles	*Miles/gal.*	*Dollars*	*Cents*	*Cents*	*Dollars*	*Cents*	*Cents*	*Dollars*	*Cents*	*Cents*
California	1946	T	26, 000	2. 58	1, 062	4. 085	0. 113	1, 847	3. 694	0. 103	1, 230	2. 460	0. 068
Colorado [3]	1950	T	50, 000	5. 1	4, 975	9. 950	. 276	5, 392	10. 784	. 299	2, 627	5. 254	. 146
Colorado [4]	1950	T	50, 000	5. 1	4, 779	9. 558	. 265	5, 098	10. 196	. 283	-----	-----	-------
Idaho	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	2, 491	4. 982	. 138
Illinois	1948	T	36, 500	4. 3	1, 189	3. 249	. 090	1, 500	3. 000	. 083	1, 937	3. 874	. 108
Louisiana [5]	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	-----	-----	-------
Minnesota [5]	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	-----	-----	-------
New York [5]	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	-----	-----	-------
Ohio	1951	S	45, 500	2. 3	2, 568	5. 644	. 157	2, 310	4. 620	. 128	2, 542	5. 084	. 141
Ohio	1953	I	47, 900	2. 6	2, 303	4. 808	. 134	2, 118	4. 236	. 118	-----	-----	-------
Oregon	--------	--------	--------	--------	-----	-----	-------	-----	-----	-------	3, 225	6. 450	. 179
Utah	1950	T	42, 000	4. 45	1, 562	3. 719	. 103	1, 825	3. 650	. 101	1, 165	2. 330	. 065
Washington	1948	T	26, 333	3. 1	1, 354	5. 142	. 143	1, 766	3. 532	. 098	1, 446	2. 892	. 080

[1] Method used to allocate tax responsibility indicated thus: I=Incremental; S=Standard cost; T=Ton mile.

[2] Travel, 50,000 miles per year; fuel consumption rate, 3.4 miles per gallon.

[3] Motor-fuel tax rate of 8.5 cents per gallon and low registration fees.

[4] Motor-fuel tax rate of 6.5 cents per gallon and high registration fees.

[5] This vehicle combination not permitted by State size and weight regulations.

Appendix B

Following is a reference list of the reports of State highway finance and taxation studies that were consulted in the preparation of this article. Eleven studies in nine States are included. In Ohio the study made by H. D. Simpson was based on the gross ton-mile concept; that by D. F. Pancoast was a solution by the incremental method. In Washington there was only one study, utilizing the gross ton-mile method; but two reports were prepared successively by James C. Nelson.

Of the numerous other State highway finance studies available, some were too far removed in time, and others afforded no clear-cut basis for comparison. Studies now in progress, notably the incremental studies in Washington, Kentucky, and Montana, should provide further comparative data.

CALIFORNIA

A proposed system of highway financing for the State of California, by Bertram H. Lindman. A report submitted to the Joint Fact-Finding Committee of the California Legislature on Highways, Streets and Bridges. Sacramento, Nov. 1946.

COLORADO

Colorado's highway needs and highway financing. Preliminary report of the Colorado Highway Planning Committee. Denver, Oct. 1950.

ILLINOIS

A highway improvement program for Illinois, prepared by Griffenhagen and Associates for the Illinois Division of Highways. Springfield, Nov. 1948.

LOUISIANA

Financing highway improvements in Louisiana, by William D. Ross. A financial analysis for the Legislative Council. Louisiana State University, Baton Rouge, 1955.

MINNESOTA

Financing a proposed highway program in Minnesota, by the Public Administration Service. A report to the Minnesota Highway Study Commission. St. Paul, Oct. 1954.

NEW YORK

Highway finance and taxation in New York, prepared by Griffenhagen and Associates for the Citizens Public Expenditure Survey, Inc., of New York State. New York, Feb. 1950.

OHIO

Highway finance, by Herbert D. Simpson. A study prepared for the Ohio Program Commission of the Ohio Department of Highways. Columbus, Sept. 1951. *Allocation of highway costs in Ohio by the incremental method,* by D. F. Pancoast, Ohio Department of Highways. Columbus, Dec. 1953.

UTAH

Financing needs and allocating costs of highways among highway users in Utah, by the Bureau of Economics and Business Research, Utah University. Prepared for the Legislative Council of Utah. Salt Lake City, 1950.

WASHINGTON

Financing Washington's highways, roads, and streets, by Dr. James C. Nelson. A report submitted to the Joint Fact-Finding Committee on Highways, Streets and Bridges of the State of Washington. Olympia, Oct. 1948. *Taxing Washington's motor vehicles equitably for highway services,* by Dr. James C. Nelson. A report submitted to the Joint Fact. Finding Committee on Highways, Streets and Bridges of the State of Washington. Olympia, Sept. 1950

United States-Canadian Border Commercial Traffic

*BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS*

**Reported by DANIEL O'FLAHERTY
Head, Traffic and Travel Studies Unit**

In this article, the results of a study made of commercial vehicles crossing the United States-Canadian border on an average weekday in July or August of 1954 are reported. Vehicle operators interviewed at 16 stations along the border supplied the following information: origin and destination of trips, average distances traveled in each of the two countries, frequency of trips, classification of trip (private or for hire), commodity being hauled, State and/or Province of vehicle registration, and licensing authority of the operator's permit.

The weekday commercial travel at the 16 stations approximated three-fourths of the annual average daily commercial traffic crossing the border on all highways connecting the two countries. The average daily commercial traffic for July and August was found to be about the same as the annual average daily volume.

Approximately two-thirds of the commercial vehicles crossing the border were registered in Canada, whereas two-thirds of the mileage traveled was in the United States. The majority of these vehicles were traveling between Provinces and States adjacent to the border. At some border stations many of the trips were very short, being less than 5 miles in length (one way). There were, however, nearly 200 trips which were over 1,000 miles in length and at least 68 which were over 2,000 miles. Four trips of the latter group were estimated to be more than 3,000 miles. The average trip distance for all commercial vehicles crossing the border was 251 miles of which 169 miles involved travel in the United States and 82 miles in Canada.

Single-unit trucks accounted for 56 percent of the trips and 25 percent of the mileage traveled, and truck combinations with 37 percent of the trips accounted for 69 percent of the travel. The remaining 7 percent of trips and 6 percent of travel was by buses.

About one-third of the travel was by vehicles classed as private carriers and two-thirds by vehicles classed as for-hire carriers. A comparison of truck body types indicates that stake or platform bodies were the most numerous, closely followed by van and box-type bodies.

Approximately one-eighth of the trips across the border were nonrepetitive or very infrequent, whereas over half were repeated more often than once a week, and about a fifth were more often than once a day.

About half of all trucks were carrying loads. Two-thirds of the loaded vehicles carried manufactured and miscellaneous products. No other single classification of products was hauled by as many as 10 percent of the vehicles, although vehicles hauling agricultural and animal products each approached that figure.

Less than 200 operators of the 3,093 interviewed in this study reported that they were licensed to drive by both State and Province authorities.

AT the request of the Department of State and in cooperation with that Department and other agencies of the Federal Government, the Bureau of Public Roads undertook a study of the commercial traffic crossing the United States-Canadian border. The field work was done by the highway departments of 8 of the 12 States bordering on Canada as a part of the cooperative highway planning survey program carried out jointly by the States and the Bureau of Public Roads.

Information was obtained regarding truck and bus travel on a typical summer weekday at 16 stations located along the border. Roadside interviews were made at these locations on a weekday in July or August 1954. The drivers of all commercial vehicles passing the interview stations during a 24-hour period were interviewed except in Maine where only 76 percent of the drivers at the 3 stations in the State were interviewed. The analysis of the border traffic was limited to the actual sample. It would be necessary, therefore, to increase the Maine figures about 32 percent if it were desired to compare the number of crossings on the Maine border with the number of crossings at other border States.

Information Obtained at Border Stations

Operators of commercial vehicles were questioned concerning the origin and destination of trip, the commodity hauled (when not obvious), the trip distance in the United States and Canada separately, the States and/or Provinces of vehicle registration, the classification of trip ("private" or "for hire"), the frequency of trips across the border, and the licensing authority of the driving permit.

In the study there were 3,093 vehicle records obtained of which 222 were for buses and 2,871 for trucks. This number amounted to more than three-fourths of the daily commercial traffic crossing the border at all points for the summer period and for the whole year on the basis of Canadian ports of entry data.

The average daily commercial traffic crossing the border during the summer months of July and August 1954 combined, based on reports published by the Dominion Bureau of Statistics of Canada,[1] was about the same as for an average day of the year; that is, 3,950, and 4,028, respectively. To arrive at these amounts, the data included in the Canadian report were converted from monthly and annual figures to average daily traffic volumes and multiplied by two in order to account for vehicles leaving Canada as well as those entering.

The fact that more than three-fourths of the average daily traffic at all points was intercepted at 16 locations on weekdays indicates that an adequate sample was obtained for the border as a whole. That is true, notwithstanding that the number of vehicles crossing the border for certain Provinces as shown in the Canadian report vary considerably in volume from that reported in this article. The travel between individual Provinces and States having many minor roads crossing the border was not as well represented in the sample as the travel for the whole border, which was the subject of this study.

Trip Origins and Destinations

A basic fact developed from this study is the division of registration and travel for commercial vehicles between the two countries. This is illustrated in figure 1 which shows that almost two-thirds of the home country registration is Canadian, yet slightly more than

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[1] *Travel between Canada and other countries 1954*, by the Dominion Bureau of Statistics, International Trade Division, Balance of Payments Section. Ottawa. 1955. Tables 13 and 17, pp. 40 and 45.

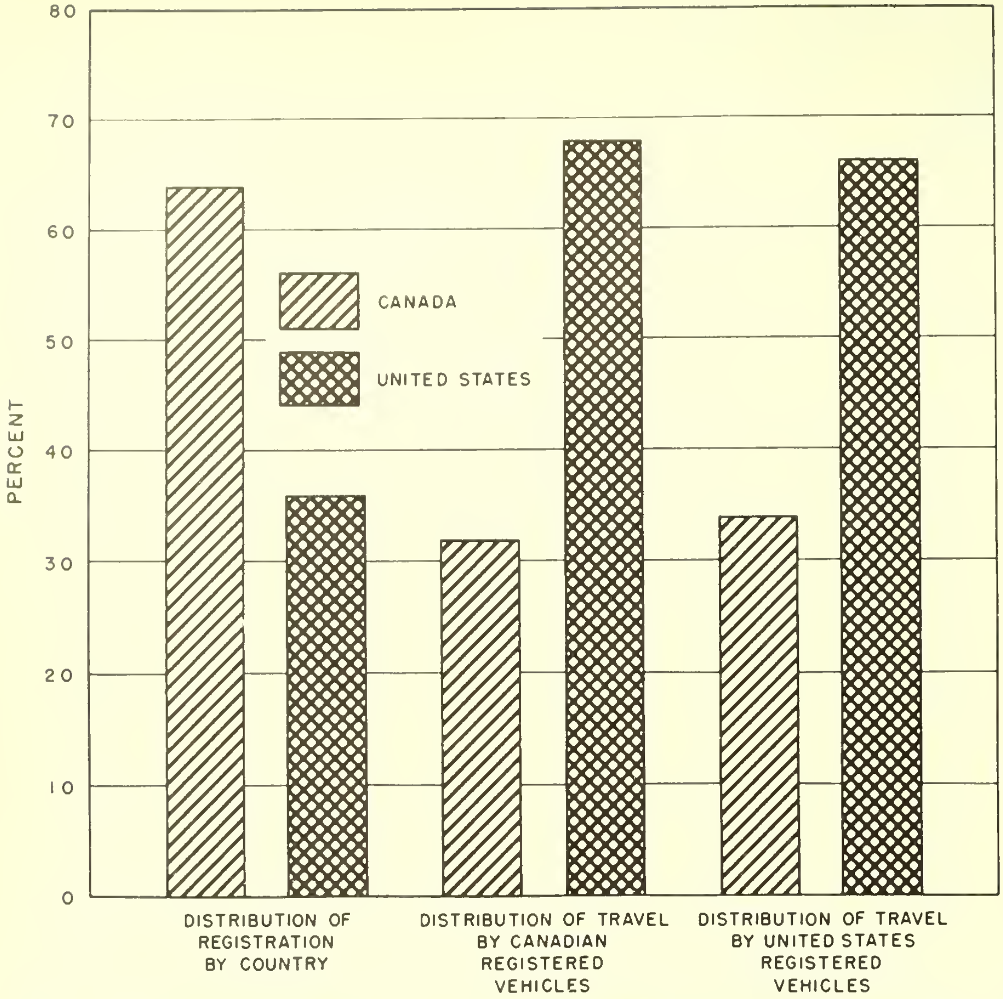


Figure 1.—Percentage distribution of commercial vehicles crossing the border on an average summer weekday in 1954, according to country of registration and area of travel.

two-thirds of the travel is in the United States.

Travel between the United States and Canada involved trips with origin or destination in 35 States, Alaska, and 9 Provinces. Most of the trips were between adjacent States and Provinces; there were, however, some very long trips of which at least four exceeded 3,000 miles in length.

Table 1 shows the origin and destination of all trips recorded in this study. Ontario was the origin or destination of more trips (1,585) than any other Province or State, and 13 of these trips had both origin and destination in Ontario. Michigan was the origin or destination of more trips than any other State with 1,079 trips, followed by New York and Maine with 620 and 536 trips, respectively. Approximately 88 percent of the trips across the border originated or terminated in a border State.

The 886 vehicles recorded as traveling between Ontario and Michigan are the highest number between any specific State and Province. There were 413 vehicles traveling between New Brunswick and Maine, 334 between Ontario and New York, 143 between Ontario and Minnesota, 134 between Quebec and Vermont, and 133 between British Columbia and Washington.

By studying table 2 in conjunction with table 1, it is found that there were 218 trips with both origins and destinations within the United States of which 158 were between Michigan and New York. These vehicles passed through Canada and since the more important Michigan and New York border crossings were in operation in this study, it is probable that most of the 158 vehicles were duplicated in the reporting at stations 6, 7, or 8 and at stations 9, 10, or 11.

There were 190 trips with both origin and destination in Canada. Had all the stations on the border been operated, these trips would need to be divided by two since they would have crossed the border twice. However, an analysis of the detailed origin and destination as well as the mileage traveled in the United States and Canada definitely established that a considerable number of these trips were not duplicated.

Of the 3,093 trips across the border, no more than 204 [(218+190)÷2=204] could be duplicated trips and, as stated, many of these were not. Five of the United States-to-United States trips were between Alaska and the States, and four were between the mainland of Washington and Point Roberts, Wash., on the tip of a peninsula that can be reached by highway from the United States only by passing through Canada. The five Alaska trips would involve only one crossing and it is known that the four Washington trips were recorded but once. Therefore, the relatively few duplicated trips included in this study have not been eliminated or adjusted because they involved so small a percentage of the sample and lacking complete information on crossings at all border stations, it could not be definitely determined which trips should be eliminated.

Distribution of Travel by Border Stations

The number of commercial vehicles crossing the border at each of the 16 stations is shown in table 2 and figure 2. For example, 189 vehicles crossed at station 1 in Washington of which 83 were registered in the United States and 106 were registered in Canada.

Figures 3–8 show origins and destinations of trips between States and Provinces by stations. Code numbers beside the line to a State or Province show the termini of trips with the number of trips indicated at the end of the line. For example, the codes 04–62 in figure 3 along with the numeral 5 at each end of the line indicate that there were five California-British Columbia trips recorded at station 1.

Travel data between States and Provinces at station 1 and at stations 9 and 10 combined are shown in figure 3. For instance, there were 133 trips between Washington and British Columbia at station 1, and 333 trips between New York and Ontario at stations 9 and 10. Note the two very long trips between California and Ontario, the four trips between Michigan and British Columbia, and also the seven trips between Ontario and British Columbia via the United States.

Trips involving crossings at station 2 and at stations 12 and 13 combined are shown in figure 4. A long trip between Texas and Quebec was recorded at the border station in Vermont, and three vehicles traveling between Texas and Alberta crossed at the Montana border station. Other long trips through station 2 were between Alaska and Montana, Illinois, and Wisconsin. Here are found 22 Canada-to-Canada trips between Ontario and Alberta via the United States.

Trips across the border at station 3 i North Dakota and at station 11 in New Yor are shown in figure 5 (p. 90). Traffic at thes locations is of relatively low volume. An unusually long trip between California an Quebec was recorded at station 11.

Trips recorded at stations 4 and 5 in Minnesota and 14 and 15 in Maine are presented i figure 6 (p. 90). Except for station 4, most o the trips at these locations were between adjacent States and Provinces. Although 14 trips (table 2) were recorded at station 5 an only 110 trips at station 4, the termini distribution was more varied at station 4.

Trips recorded at stations 6 and 16 ar shown in figure 7 (p. 91). At station 6, Por Huron, Mich., there were 36 trips betwee Michigan and New York via Canada. Ther were also six trips between Alberta and Ontari and eight trips between Manitoba and Ontario by way of the United States. Singl

Table 1.—Origin and destination and number of trips made by commercial vehicles crossing the Canadian-United States border on an average summer weekday in 1954

Origin of trip	Destination of trip																												
	Province										Border States													Nonborder States, by regions					
	Alberta	British Columbia	Manitoba	New Brunswick	Nova Scotia	Ontario	Prince Edward Island	Quebec	Saskatchewan	Total	Idaho	Maine	Michigan	Minnesota	Montana	New Hampshire	New York	North Dakota	Ohio	Vermont	Washington	Wisconsin	Total	Eastern States [1]	Central States [2]	Western States [3]	Total	Territory of Alaska	Total trips
Province:																													
Alberta	--	2	4	---	--	19	---	1	--	26	--	---	1	---	52	---	4	--	--	--	1	2	60	1	3	6	10	---	96
British Columbia	16	6	2	---	--	8	---	1	1	34	--	---	1	1	1	---	---	--	--	--	72	-	75	--	1	7	8	---	117
Manitoba	3	3	--	---	--	28	---	4	--	38	--	---	---	27	--	---	1	7	--	--	--	-	35	--	4	--	4	---	77
New Brunswick	--	--	--	3	--	---	---	1	--	4	--	224	---	---	--	---	---	--	--	--	--	-	224	8	--	--	8	---	236
Nova Scotia	--	--	--	---	--	---	---	---	--	-----	--	10	---	---	--	---	---	--	--	--	--	-	10	9	--	--	9	---	19
Ontario	22	4	23	---	--	13	---	---	12	74	--	---	447	78	--	---	188	--	12	--	2	2	729	13	6	1	20	---	823
Prince Edward Island	--	--	--	---	--	---	---	1	--	1	--	---	---	---	--	---	---	--	--	--	--	-	-----	2	--	--	2	---	3
Quebec	1	--	2	4	--	---	---	---	--	7	--	50	1	---	--	3	52	--	--	86	--	-	192	25	2	--	27	---	226
Saskatchewan	--	1	--	---	--	5	---	---	--	6	--	---	---	---	1	---	---	17	--	--	--	-	18	--	1	--	1	---	25
Total	42	16	31	7	--	73	---	8	13	190	--	284	450	106	54	3	245	24	12	86	75	4	1,343	58	17	14	89	---	1,622
Border States:																													
Idaho	--	--	--	---	--	1	---	---	--	1	--	---	---	---	--	---	---	--	--	--	--	-	-----	--	--	--	---	----	1
Maine	--	--	--	189	4	---	---	40	--	233	--	17	2	---	--	---	---	--	--	--	--	-	19	--	--	--	---	---	252
Michigan	1	3	--	---	--	439	---	3	1	447	--	---	---	---	--	---	98	--	--	--	--	-	98	10	--	--	10	---	555
Minnesota	--	--	17	---	--	65	---	---	1	83	--	---	---	---	--	---	---	--	--	--	--	-	-----	--	--	--	---	---	83
Montana	40	--	--	---	--	---	---	---	--	40	--	---	---	---	--	---	---	--	--	--	--	-	-----	--	--	--	---	2	42
New Hampshire	--	--	--	---	--	1	---	3	--	4	--	---	---	---	--	---	---	--	--	--	--	-	-----	--	1	--	1	---	5
New York	--	--	--	---	--	146	---	64	--	210	--	---	60	---	--	---	---	--	--	--	--	-	60	--	5	--	5	---	275
North Dakota	1	1	7	---	--	1	---	---	8	18	--	---	---	---	--	---	---	--	--	--	--	-	-----	--	--	--	---	---	18
Ohio	--	--	--	---	--	16	---	---	--	16	--	---	---	---	--	---	---	--	--	--	--	-	-----	--	--	--	---	---	16
Vermont	--	--	--	---	--	---	---	48	--	48	--	---	---	---	--	---	---	--	--	--	--	-	-----	--	--	--	---	---	48
Washington	--	61	--	---	--	1	---	---	--	62	--	---	---	---	--	---	---	--	--	--	4	-	4	--	--	--	---	---	66
Wisconsin	--	--	1	---	--	1	---	---	--	2	--	---	---	---	--	---	---	--	--	--	--	-	-----	--	--	--	---	1	3
Total	42	65	25	189	4	671	---	158	10	1,164	--	17	62	---	--	---	98	--	--	--	4	-	181	10	6	--	16	3	1,364
Nonborder States, by regions:																													
Eastern States [1]	--	--	--	7	9	16	4	18	--	54	--	---	12	---	--	---	---	--	--	--	--	1	13	--	1	--	1	---	68
Central States [2]	2	--	3	---	--	12	---	2	5	24	--	---	---	---	--	---	1	--	--	--	--	-	1	--	--	--	---	1	26
Western States [3]	4	3	--	---	--	3	---	1	--	11	--	---	---	---	--	---	1	--	--	--	--	-	1	--	--	--	---	---	12
Total	6	3	3	7	9	31	4	21	5	89	--	---	12	---	--	---	2	--	--	--	--	1	15	--	1	--	1	1	106
Territory of Alaska	--	--	--	---	--	---	---	---	--	-----	--	---	---	---	1	---	---	--	--	--	--	-	1	--	--	--	---	---	1
Total, all trips	90	84	59	203	13	775	4	187	28	1,443	--	301	524	106	55	3	345	24	12	86	79	5	1,540	68	24	14	106	4	3,093

[1] Includes the following States: Connecticut, Maryland, Massachusetts, New Jersey, Pennsylvania, Rhode Island, South Carolina, and Virginia.

[2] Includes the following States: Alabama, Arkansas, Illinois, Indiana, Iowa, Kentucky, Missouri, Nebraska, Oklahoma, and Texas.

[3] Includes the following States: California, Colorado, Nevada, Oregon, and Wyoming.

trips between Oklahoma and Ontario and South Carolina and Ontario were also recorded. At station 16 in Maine, 183 trips were recorded between Maine and New Brunswick. Only three States other than Maine were involved in trips at station 16.

The greatest number of trips across the border at any one general location were recorded at Detroit (stations 7–8). Travel data for the two stations are combined in figure 8 (p. 91). Of the 938 vehicle operators interviewed at Detroit, 784 were traveling between Michigan and Ontario. Including crossings at Port Huron (station 6; fig. 7), there were 886 trips between Michigan and Ontario. The total United States-to-United States and Canada-to-Canada trips recorded at stations 7–8 exceeded that of any other border area. Here there were 51 Canada-to-Canada trips of which 22 were between Manitoba and Ontario, and 56 United States-to-United States trips of which 39 involved travel between Michigan and New York. At stations 9–10 combined (fig. 3) there were 98 United

Table 2.—Number of commercial vehicles crossing the border at 16 stations on an average summer weekday in 1954, classified by country of vehicle registration and country of origin and destination

Station number	Location of station	Origin and destination of travel of United States registered vehicles					Origin and destination of travel of Canadian registered vehicles					Origin and destination of travel of all vehicles				
		Canada to Canada	Canada to United States	United States to Canada	United States to United States	Total	Canada to Canada	Canada to United States	United States to Canada	United States to United States	Total	Canada to Canada	Canada to United States	United States to Canada	United States to United States	Total
1	Blaine, Wash	1	35	43	4	83	32	49	25	-----	106	33	84	68	4	189
2	Sweetgrass, Mont	-----	21	15	5	41	29	49	33	-----	111	29	70	48	5	152
3	Portland, N. Dak	-----	11	11	-----	22	4	8	2	-----	14	4	19	13	-----	36
4	Noyes, Minn	6	18	15	-----	39	36	20	15	-----	71	42	38	30	-----	110
5	International Falls, Minn	-----	14	7	-----	21	4	62	58	-----	124	4	76	65	-----	145
6	Port Huron, Mich	2	21	23	36	82	14	41	28	1	84	16	62	51	37	166
7	Detroit Tunnel, Mich	-----	43	42	5	90	4	138	164	-----	306	4	181	206	5	396
8	Detroit Bridge, Mich	1	52	56	46	155	46	179	157	5	387	47	231	213	51	542
9	Buffalo, N. Y	1	28	59	86	174	1	128	85	-----	214	2	156	144	86	388
10	Rainbow Bridge, Niagara Falls, N. Y	-----	3	9	9	21	-----	42	20	3	65	-----	45	29	12	86
11	Champlain, N. Y	-----	26	33	1	60	-----	37	33	-----	70	-----	63	66	1	130
12	Swanton, Vt	-----	21	25	-----	46	-----	18	15	-----	33	-----	39	40	-----	79
13	Derby Line, Vt	-----	26	19	-----	45	1	38	9	-----	48	1	64	28	-----	93
14	Jackman, Maine	-----	6	4	17	27	-----	42	30	-----	72	-----	48	34	17	99
15	Madawaska, Maine	-----	67	63	-----	130	-----	60	48	-----	108	-----	127	111	-----	238
16	Calais, Maine	4	40	33	-----	77	4	89	74	-----	167	8	129	107	-----	244
----	Total	15	432	457	209	1,113	175	1,000	796	9	1,980	190	1,432	1,253	218	3,093

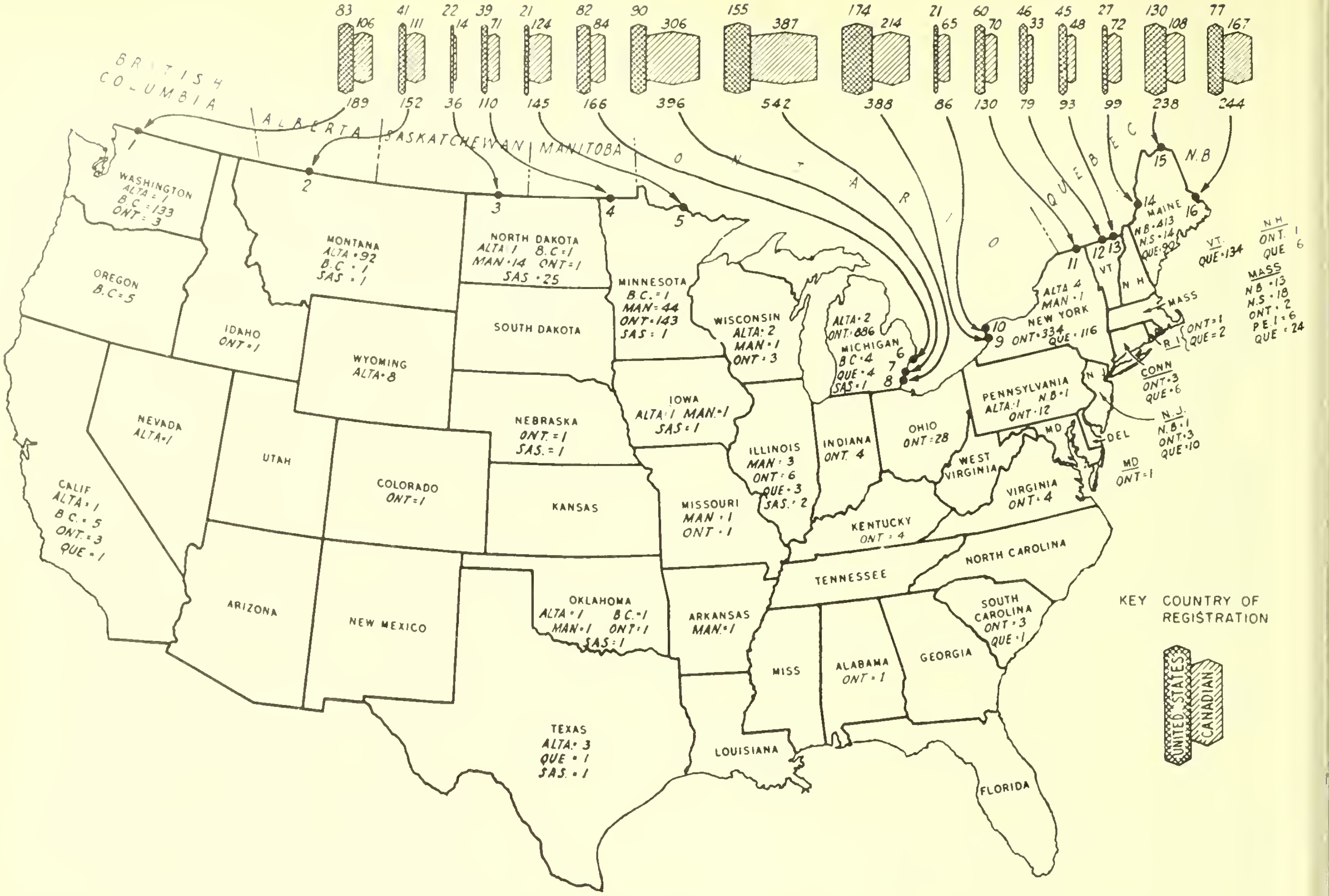

83 106 189
41 111 152
22 14 36
39 71 110
21 124 145
82 84 166
90 306 396
155 387 542
174 214 388
21 65 86
60 70 130
46 33 79
45 48 93
27 72 99
130 108 238
77 167 244
BRITISH COLUMBIA
ALBERTA
SASKATCHEWAN
MANITOBA
ONTARIO
QUEBEC
N.B
WASHINGTON ALTA = 1 B.C. = 133 ONT = 3
MONTANA ALTA = 92 B.C. = 1 SAS = 1
NORTH DAKOTA ALTA 1 B.C. = 1 MAN = 14 ONT = 1 SAS = 25
MINNESOTA B.C. = 1 MAN = 44 ONT = 143 SAS = 1
WISCONSIN ALTA = 2 MAN = 1 ONT = 3
MICHIGAN ALTA = 2 ONT = 886 B.C. = 4 QUE = 4 SAS = 1
NEW YORK ALTA 4 MAN = 1 ONT = 334 QUE = 116
MAINE NB = 413 N.S = 14 QUE = 90
VT. QUE = 134
N.H. ONT. 1 QUE 6
MASS NB = 13 N.S. = 18 ONT = 2 P.E.I. = 6 QUE = 24
R.I. ONT = 1 QUE = 2
CONN ONT = 3 QUE = 6
N.J. N.B. = 1 ONT = 3 QUE = 10
MD ONT = 1
DEL
PENNSYLVANIA ALTA = 1 N.B. = 1 ONT = 12
OHIO ONT = 28
WEST VIRGINIA
VIRGINIA ONT = 4
INDIANA ONT 4
ILLINOIS MAN = 3 ONT = 6 QUE = 3 SAS = 2
IOWA ALTA = 1 MAN = 1 SAS = 1
MISSOURI MAN = 1 ONT = 1
KENTUCKY ONT = 4
TENNESSEE
NORTH CAROLINA
SOUTH CAROLINA ONT = 3 QUE = 1
GEORGIA
ALABAMA ONT = 1
MISS
LOUISIANA
FLORIDA
ARKANSAS MAN = 1
OKLAHOMA ALTA = 1 B.C. = 1 MAN = 1 ONT = 1 SAS = 1
TEXAS ALTA = 3 QUE = 1 SAS = 1
NEW MEXICO
ARIZONA
KANSAS
NEBRASKA ONT = 1 SAS = 1
COLORADO ONT = 1
UTAH
NEVADA ALTA = 1
CALIF ALTA = 1 B.C. = 5 ONT = 3 QUE = 1
WYOMING ALTA = 8
IDAHO ONT = 1
OREGON B.C. = 5
SOUTH DAKOTA
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16
KEY COUNTRY OF REGISTRATION
UNITED STATES
CANADIAN

Figure 2.—Number of commercial vehicles crossing the border at 16 stations on an average summer weekday in 1954, according to country of registration.

States-to-United States trips of which 82 were between Michigan and New York, but only 2 trips with origin and destination in Canada were recorded.

The 39 Michigan-New York trips recorded at stations 7–8 plus the 36 such trips at station 6 (fig. 7) nearly equal the 82 trips having the same termini recorded at New York stations 9–10 (fig. 3). This of course is a duplication of sampling.

Major Portion of Travel in the United States

The average lengths of trips for major classes of commercial vehicles, according to country of vehicle registration and area of travel, are shown in figure 9 (p. 92). Except for Canadian registered buses, the average distance traveled in the United States substantially exceeds that in Canada. This is true regardless of country of vehicle registration. The average length of trip in the United States for all truck combinations was considerably more than twice the distance traveled in Canada. For all vehicles, the portion of the trip in the United States averaged 169 miles as compared with 82 miles in Canada.

Identical information to that shown in figure 9 is provided in table 3 (p. 93), but the latter contains more detailed data on average trip distances for the various types of vehicles as well as average trip distances according to class of operation.

The term "class of operation" indicates whether a commercial vehicle is operated as a "private" or "for-hire" carrier. Actually, three classes—private, contract carrier, and common carrier—were recorded in the study, but distinction between the latter two classes is difficult to obtain from the driver during the limited time available at roadside interview stations. The three terms may be briefly defined as follows:

A *private vehicle* is one which is owned by an individual or company and its use is limited to the transportation of his or its own goods or products.

A *contract carrier* enters into a specific contract, written or otherwise, for the transportation of property or goods owned by another. The owner of the vehicle does not haul for the public generally and he may refuse to accept a job.

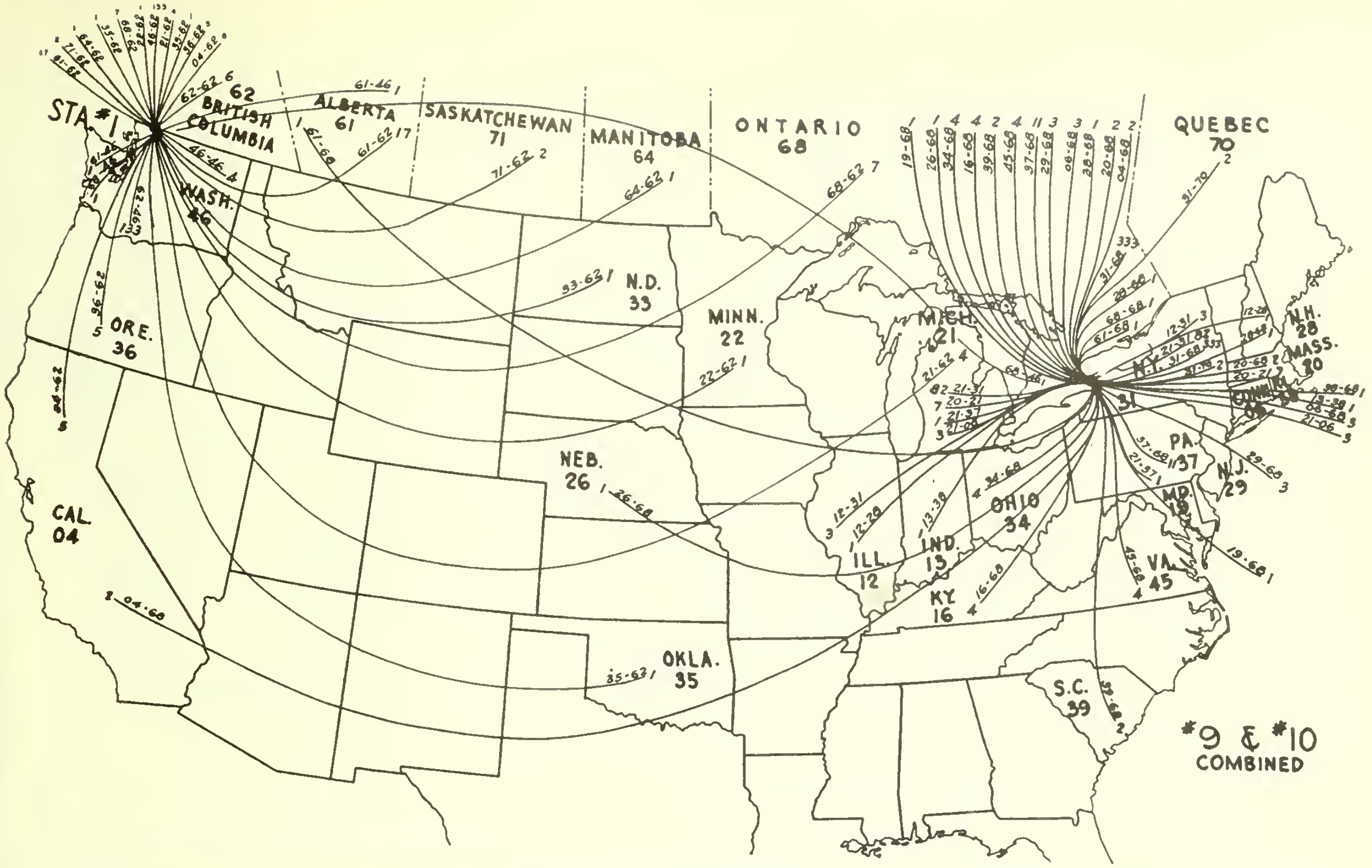
A *common carrier* hauls for the public without discrimination. His rates are published and he must accept any job that his equipment is capable of handling within the area and terms of the certificate issued.

There were so few contract-carrier vehicles recorded in this study that it was concluded that a term covering both contract- and common-carrier vehicles would better serve the purpose of this survey. Therefore, only the two major classes of private and for hire are reported here.

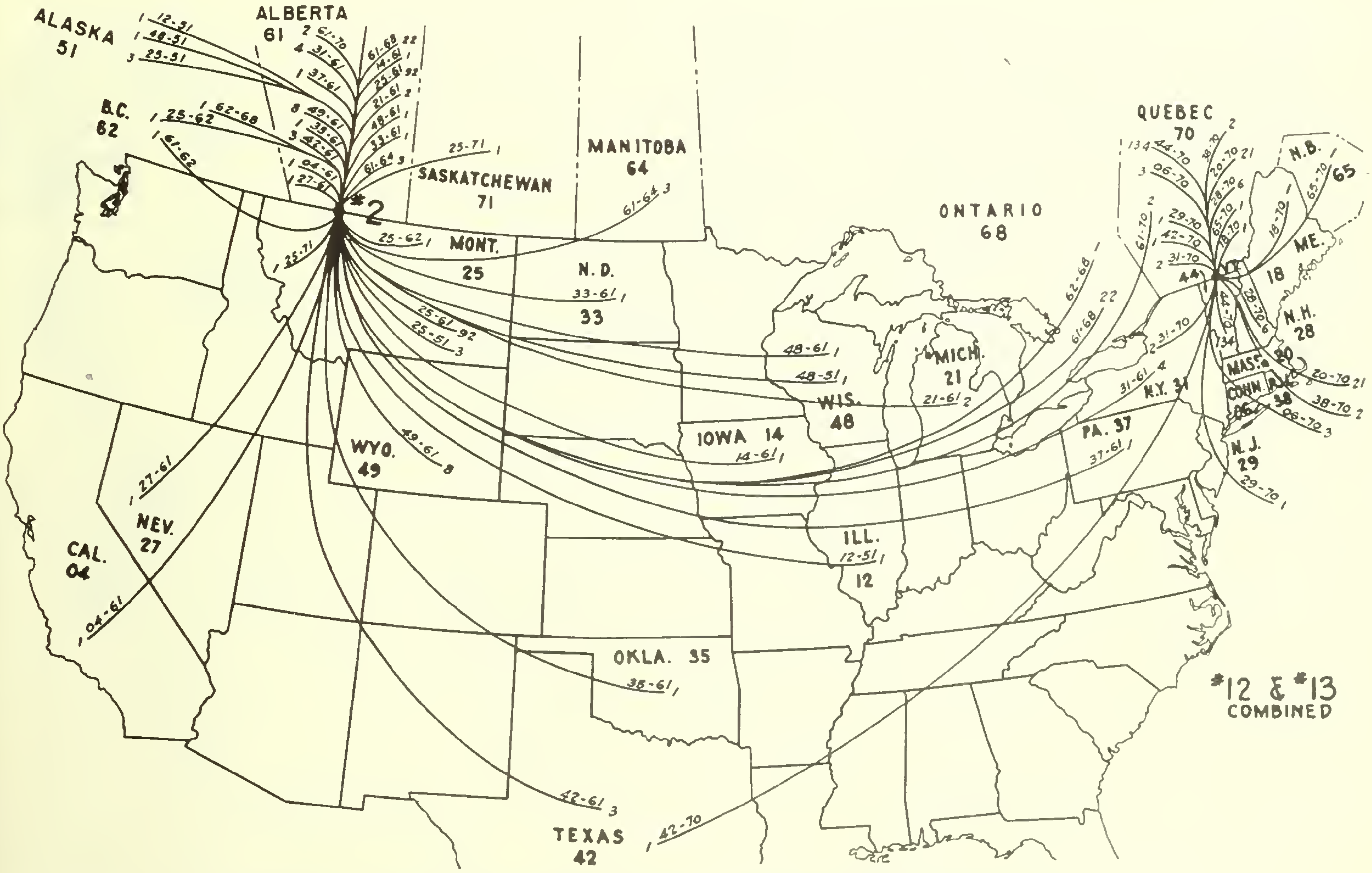
Table 3 shows that average distances traveled in the United States exceed those in Canada, except for a small percentage of single-unit for-hire trucks, 19 truck and trailer combinations of United States registry, and 124 Canadian registered buses.

The distribution of travel by private and for-hire classes of vehicle operation is similar regardless of country of vehicle registration. Privately operated vehicles accounted for 34.8 percent of the travel, and the for-hire class, 65.2 percent. When the major types of vehicles are considered separately, it is found that the relation of travel for private and for-hire classes is as follows: single-unit trucks, 88.1 and 11.9 percent; truck combinations, 17.4 and 82.6 percent; and buses, 11.5 and 88.5 percent, respectively.

Forty percent of the trucks and 19 percent of the buses crossing the border traveled less than 10 miles in each country (total trip distance less than 20 miles). Nine percent of the trucks and 7 percent of the buses traveled between 10 and 99 miles in each country. More than two-thirds of all trucks traveled less than 100 miles in each country, or stated another way, the trip distance was



igure 3.—Origin and destination and number of trips made by commercial vehicles crossing the border at station 1 and stations 9 and 10 (combined) on an average summer weekday in 1954.



igure 4.—Origin and destination and number of trips made by commercial vehicles crossing the border at station 2 and stations 12 and 13 (combined) on an average summer weekday in 1954.

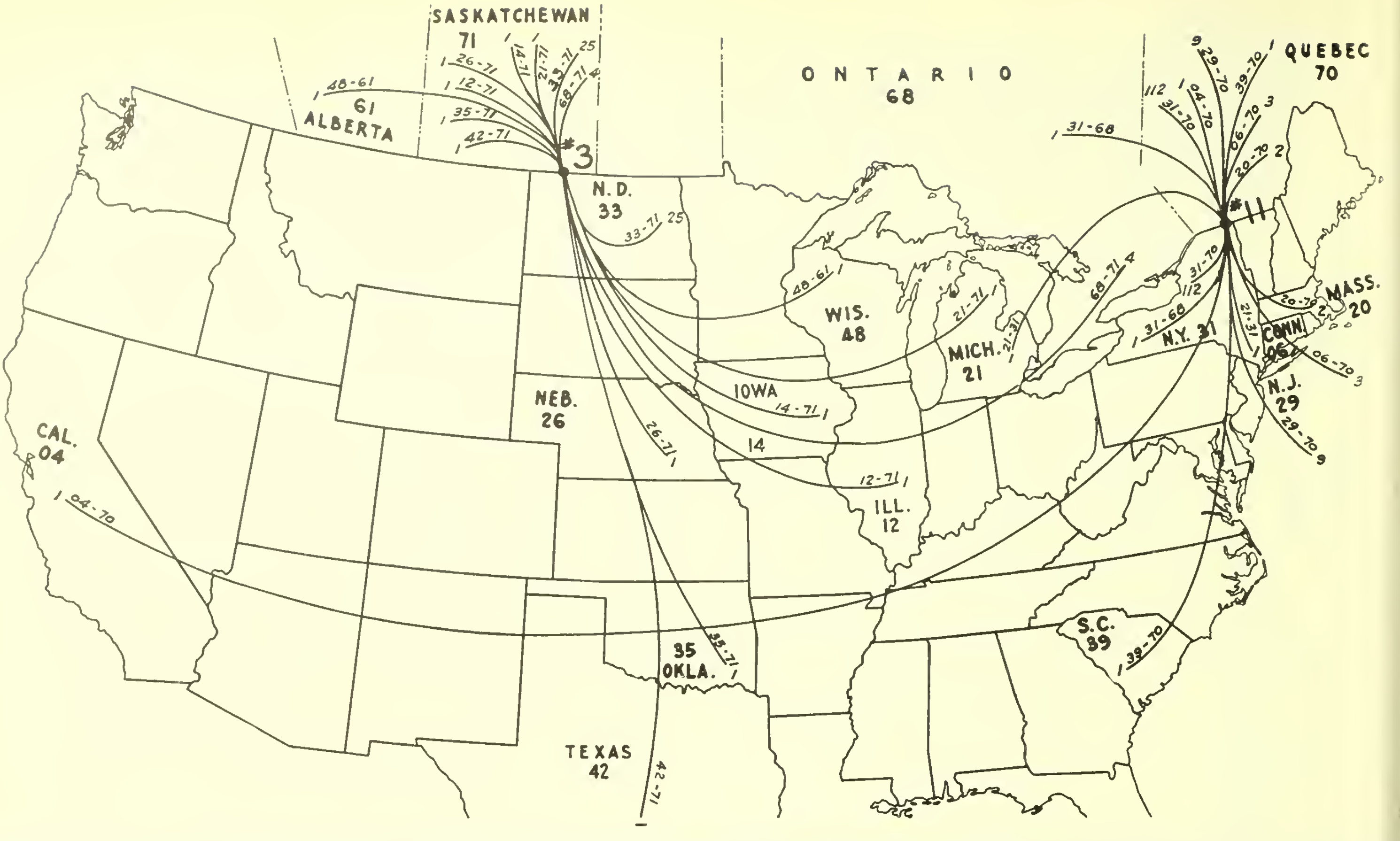


Figure 5.—Origin and destination and number of trips made by commercial vehicles crossing the border at stations 3 and 11 on an average summer weekday in 1954.

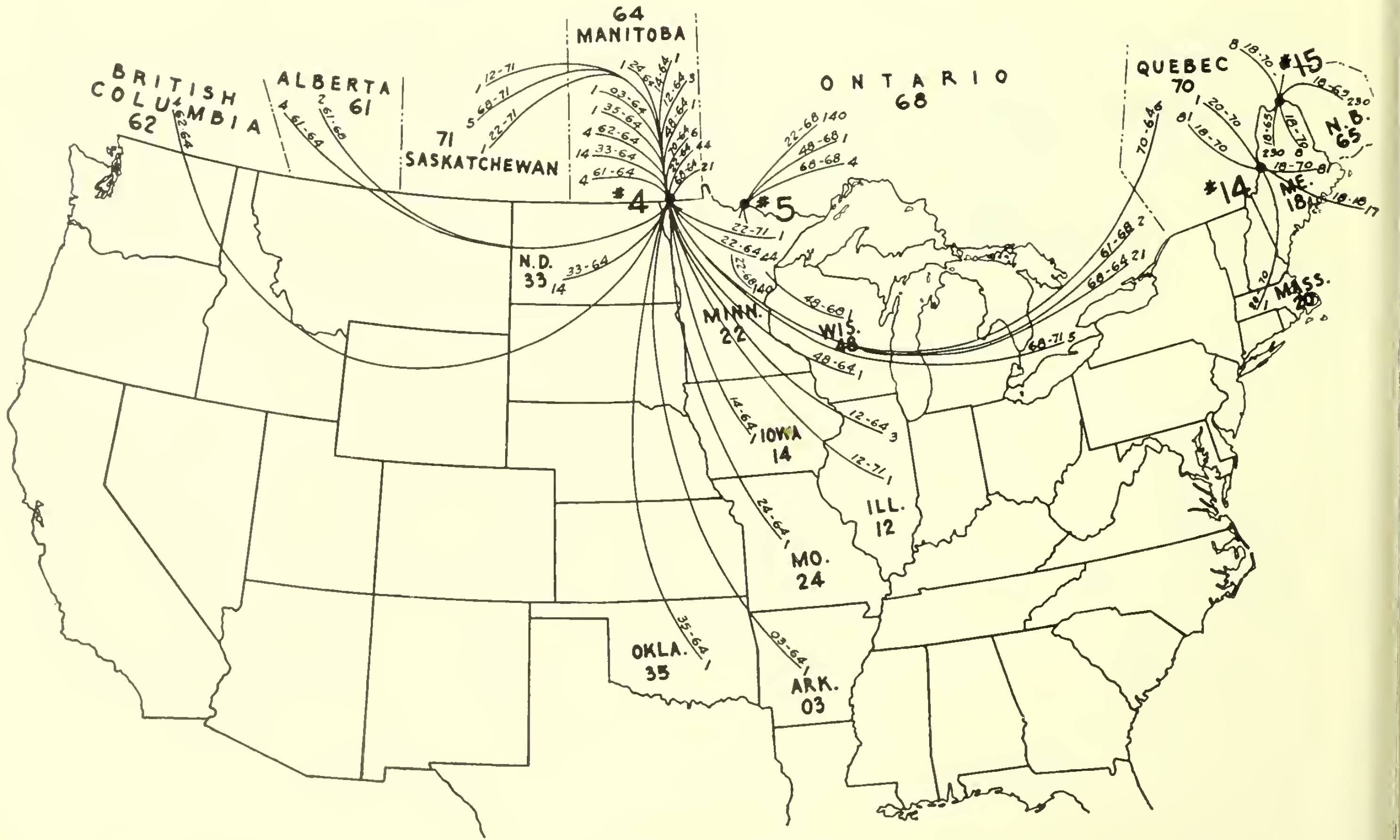
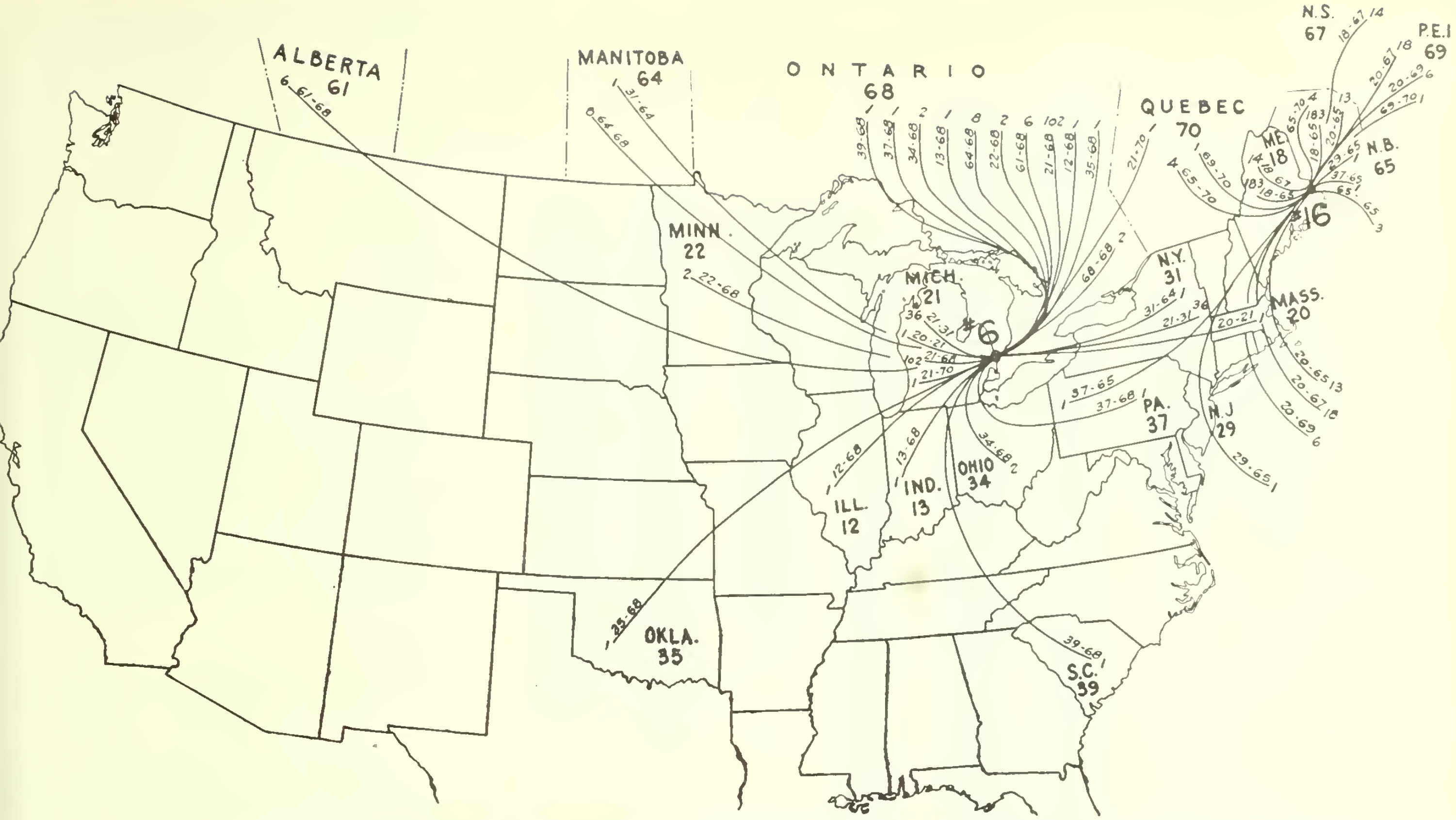
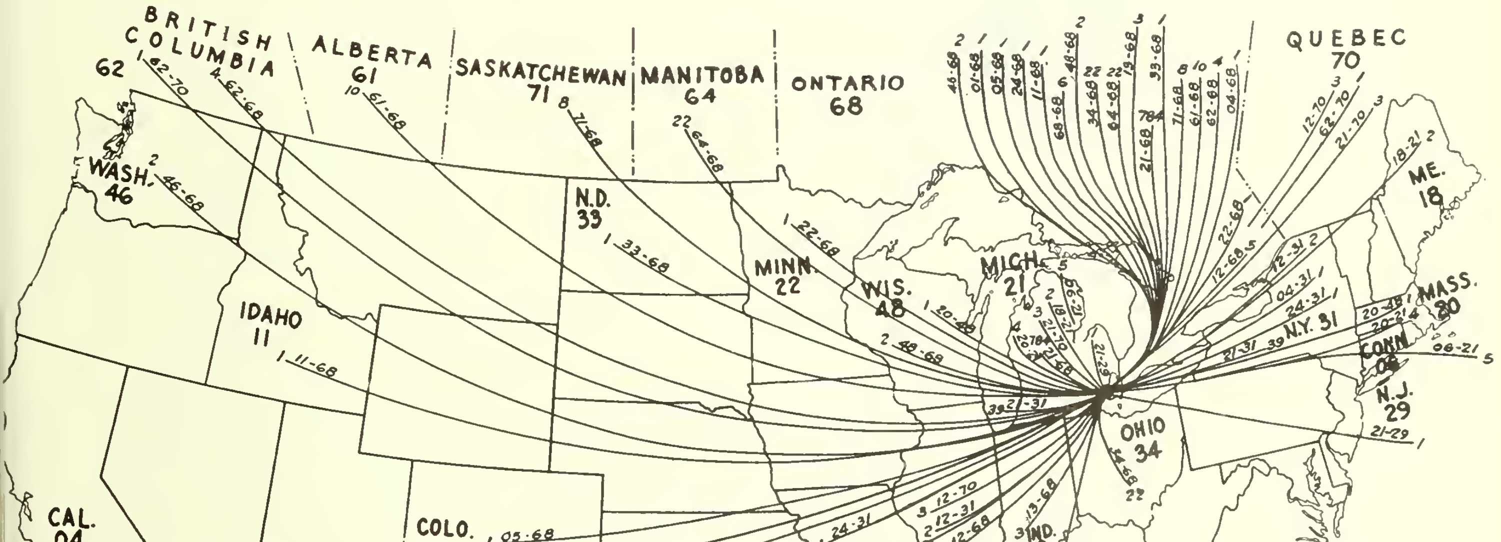


Figure 6.—Origin and destination and number of trips made by commercial vehicles crossing the border at stations 4, 5, 14, and 15 on an average summer weekday in 1954.



igure 7.—Origin and destination and number of trips made by commercial vehicles crossing the border at stations 6 and 16 on an average summer weekday in 1954.



igure 8.—Origin and destination and number of trips made by commercial vehicles crossing the border at stations 7 and 8 combined on an average summer weekday in 1954.

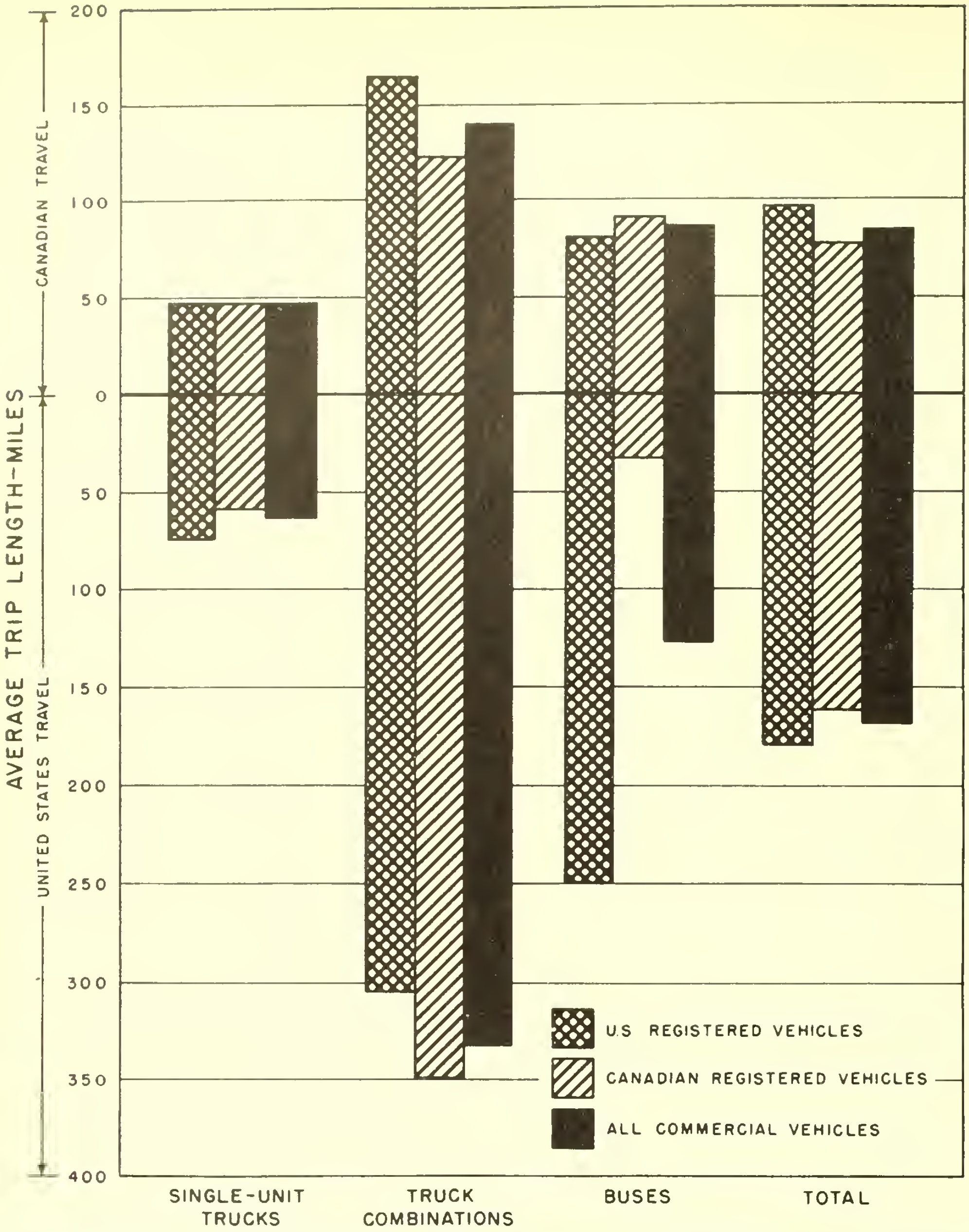


Figure 9.—Average length of trips made by commercial vehicles crossing the border on an average summer weekday in 1954, according to type of vehicle, country of registration, and area of travel.

less than 200 miles in both countries. About half (46 percent) of the buses fell in this category.

About 6 percent of the trucks and less than 1 percent of the buses traveled 1,000 miles or more in the United States. Less than 1 percent of the trucks and no buses traveled as much as 1,000 miles in Canada.

For most of the very long trips the major portion of travel was in the United States and some of these trips had both origins and destinations in Canada. One of the longest trips of the latter type was between Toronto, Ontario, and Vancouver, British Columbia, and was made by a vehicle of Canadian registration. The total length of trip was reported as 3,000 miles—300 miles in Canada and 2,700 miles in the United States. The vehicle was a 4-axle tractor-semitrailer combination hauling paper. A number of other vehicles moving from Canada to Canada traveled more than 1,000 miles in the United States. A trip of 3,032 miles between Vancouver, British Columbia, and Oklahoma City, Okla., involved 3,000 miles of travel in the United States. This vehicle was a pickup truck carrying personal luggage and was registered in Texas.

Majority of Heavy Vehicles in For-Hire Class

The percentage distribution of the number of commercial vehicles crossing the border, classified according to vehicle type, country of registration, and class of operation, is shown in table 4.

Single-unit trucks were largely operated as private carriers. Only 4.2 percent of the 2-axle, 4-tire trucks were operated as for-hire carriers, and only 17.4 percent of the 2-axle, 6-tire vehicles were reported in this category. In contrast, 64.6 percent of the 3-axle tractor-semitrailer combinations and 81.1 percent of the 4-axle tractor-semitrailer combinations were reported in the for-hire classification. Of the few truck-trailer combinations recorded in the study, 56.6 percent were in the for-hire class. For-hire operated buses accounted for 94.1 percent of the total. Of the 222 buses reported, 13 were private carriers and only 25 had a seating capacity of less than 30 passengers.

The relation of the number of private and for-hire vehicles is almost the opposite of their respective travel when all vehicles are considered as a group. This cannot be said, however, for the individual types of vehicles when they are compared separately.

The percentages of United States registered vehicles in the private and for-hire categories were 55.5 and 44.5 percent, respectively; Canadian registered vehicles in the two classes were 63.6 and 36.4 percent. For all vehicles the percentage in the privately operated class was 60.7 percent, and the for-hire class, 39.3 percent.

Truck Combinations Travel Greater Distances

In table 5, the number of vehicles and mileage traveled are shown for single-unit trucks, truck combinations, and buses. Although single-unit trucks represented 56 percent of the vehicles recorded, this type of vehicle accounted for only 25 percent of the travel.

Truck combinations amounted to less than 37 percent of the total vehicles recorded, yet these vehicles accounted for almost 69 percent of the total travel. This indicates that trip lengths are much greater for the heavier type vehicles than for the lighter types. Buses accounted for slightly more than 7 percent of the vehicles recorded and 6 percent of the travel.

The percentage of vehicles registered in the United States and Canada is compared with the percentage of travel in each country by type of vehicle in table 6. For each type of vehicle the greater proportion are registered in Canada and the greater proportion of the travel is in the United States.

This table indicates that 2-axle, 4-tire single-unit trucks, mostly panels and pickups, have a larger proportion of Canadian registration than any other type of vehicle. However, the travel by these vehicles, although greater in the United States than in Canada, accounted for a smaller percentage of total travel in the United States than any other vehicle. Only 32 percent of these vehicles were registered in the United States, yet 57 percent of the travel occurred in the United States.

Of the total number of single-unit trucks recorded, approximately 34 percent were registered in the United States and 66 percent in Canada, whereas 58 percent of the travel was in the United States and 42 percent was in Canada.

About 40 percent of the number of 4-axle tractor-semitrailer combinations crossing the border were registered in the United States but nearly three-fourths (73 percent) of the travel by vehicles of this type was in the United States.

able 3.—Number of commercial vehicles crossing the border on an average summer weekday in 1954, according to type of vehicle and class of operation, country of registration, and average length of trip [1]

Type of vehicle and class of operation	United States registered vehicles				Canadian registered vehicles				All vehicles			
	Number of vehicles	Average length of trip (in miles) traveled in—			Number of vehicles	Average length of trip (in miles) traveled in—			Number of vehicles	Average length of trip (in miles) traveled in—		
		United States	Canada	Total		United States	Canada	Total		United States	Canada	Total
Single-unit trucks:												
2-axle, 4-tire:												
Private	277	83	47	130	605	58	48	106	882	65	48	113
For hire	18	78	92	170	21	34	49	83	39	54	69	123
2-axle, 6-tire:												
Private	239	61	39	100	434	66	44	110	673	64	42	106
For hire	50	114	68	182	92	50	51	101	142	73	56	129
All single-unit trucks:												
Private	516	72	44	116	1, 039	61	46	107	1, 555	65	45	110
For hire	68	104	74	178	113	47	50	97	181	69	59	128
Total	584	(76)	(47)	(123)	1, 152	(60)	(47)	(107)	1, 736	(65)	(47)	(112)
Truck-tractor and semitrailer combinations:												
3-axle:												
Private	47	293	122	415	128	108	55	163	175	158	73	231
For hire	131	212	190	402	188	428	134	562	319	339	157	496
4-axle or more:												
Private	41	336	158	494	70	277	118	395	111	299	133	432
For hire	193	394	155	549	284	448	150	598	477	426	152	578
Truck and trailer combinations:												
Private	6	81	165	246	17	120	89	209	23	110	109	219
For hire	13	33	132	165	17	259	61	320	30	161	92	253
All combinations:												
Private	94	298	141	439	215	164	78	242	309	205	97	302
For hire	337	309	168	477	489	434	140	574	826	383	152	535
Total	431	(307)	(162)	(469)	704	(351)	(121)	(472)	1, 135	(334)	(137)	(471)
Buses:												
Private	8	583	70	653	5	3	33	36	13	360	56	416
For hire	90	219	80	299	119	34	90	124	209	114	86	200
Total	98	(249)	(79)	(328)	124	(33)	(88)	(121)	222	(128)	(84)	(212)
All vehicles:												
Private	618	113	59	172	1, 259	78	52	130	1, 877	90	54	144
For hire	495	265	139	404	721	307	118	425	1, 216	290	127	417
Total	1, 113	(181)	(94)	(275)	1, 980	(162)	(76)	(238)	3, 093	(169)	(82)	(251)

[1] Numbers in parentheses are averages.

For all truck combinations, United States egistration amounted to 38 percent and anadian registration, 62 percent. Combina- ons traveled 71 percent of their mileage in he United States and 29 percent in Canada.

ehicles Registered in Both Countries

Of the 1,113 vehicles with home registration n the United States as reported in table 7, 68 vehicles or 24.1 percent were also regis- ered in one or more Canadian Provinces. egistrations in two Provinces accounted for .7 percent of the vehicles and in three rovinces, 0.8 percent.

There were 1,980 vehicles with home regis- ration in Canada of which 841 or 42.5 per- ent were also registered in at least one State. f this total 709 vehicles, or more than one- hird, were registered in one State, 54 in two tates, 49 in three States, 19 in four States, nd 7 in five States. The remaining three ehicles were registered in six, seven, and nine tates, respectively. In total, there were ,109 or 36 percent of all vehicles that were egistered in both countries.

Truck Body Types

Classification of vehicles according to ve- icle and body types and country of registra- ion is shown in table 8. The leading body- ype classification was stake or platform with 52 vehicles. Vehicles with van or covered nd box-type bodies followed closely with 664 nd 626, respectively.

All but one of the panel body-type vehicles ecorded were single-unit trucks. Sixty-one ercent of the stake and platform body types nd 92 percent of the box-body types were single-unit trucks. Slightly more than 93 percent of the refrigerator body types and about 72 percent of the vehicles with tank-body types were truck combinations. Ninety-two percent of the dump bodies and 63 percent of the cattle-rack body types were single-unit trucks. Eight percent of the dump-type bodies and 37 percent of the cattle-rack bodies were mounted on combinations.

Buses were included in the "other" classification along with 188 trucks of miscellaneous body types. Generally the distribution of vehicles by body type was found to be similar regardless of country of registration.

Trip Frequencies

The frequency of trips across the border between the same origins and destinations,

Table 4.—Percentage distribution of commercial vehicles crossing the border on an average summer weekday in 1954, according to type of vehicle, country of registration, and class of operation

Type of vehicle	United States registered vehicles			Canadian registered vehicles			All vehicles		
	Private	For hire	Total	Private	For hire	Total	Private	For hire	Total
Single-unit trucks:									
2-axle, 4-tire	93. 9	6. 1	100. 0	96. 6	3. 4	100. 0	95. 8	4. 2	100. 0
2-axle, 6-tire	82. 7	17. 3	100. 0	82. 5	17. 5	100. 0	82. 6	17. 4	100. 0
All single-unit trucks	88. 4	11. 6	100. 0	90. 2	9. 8	100. 0	89. 6	10. 4	100. 0
Truck-tractor and semitrailer combinations:									
3-axle	26. 4	73. 6	100. 0	40. 5	59. 5	100. 0	35. 4	64. 6	100. 0
4-axle or more	17. 5	82. 5	100. 0	19. 8	80. 2	100. 0	18. 9	81. 1	100. 0
Truck and trailer combinations	31. 6	68. 4	100. 0	50. 0	50. 0	100. 0	43. 4	56. 6	100. 0
All combinations	21. 8	78. 2	100. 0	30. 5	69. 5	100. 0	27. 2	72. 8	100. 0
Buses	8. 2	91. 8	100. 0	4. 0	96. 0	100. 0	5. 9	94. 1	100. 0
All commercial vehicles	55. 5	44. 5	100. 0	63. 6	36. 4	100. 0	60. 7	39. 3	100. 0

Table 5.—Number and percentage distribution of commercial vehicles crossing the border on an average summer weekday in 1954, according to type of vehicle and mileage of travel

Type of vehicle	Vehicles		Travel	
	Number	Percent	Miles	Percent
Single-unit trucks	1, 736	56. 1	194, 866	25. 1
Truck combinations	1, 135	36. 7	534, 890	68. 8
Buses	222	7. 2	47, 155	6. 1
Total	3, 093	100. 0	776, 911	100. 0

Table 6.—Percentage distribution of commercial vehicles crossing the border on an average summer weekday in 1954, according to type of vehicle, country of registration, and area of travel

Type of vehicle	Percent registered in—			Percent of travel in—		
	United States	Canada	Total	United States	Canada	Total
Single-unit trucks:						
2-axle, 4 tire	32.0	68.0	100.0	57.2	42.8	100.0
2-axle, 6 tire	35.5	64.5	100.0	59.4	40.6	100.0
All single-unit trucks	33.6	66.4	100.0	58.2	41.8	100.0
Truck-tractor and semitrailer combinations:						
3-axle	36.0	64.0	100.0	68.4	31.6	100.0
4-axle or more	39.8	60.2	100.0	73.1	26.9	100.0
Truck and trailer combinations	35.8	64.2	100.0	58.3	41.7	100.0
All combinations	38.0	62.0	100.0	71.0	29.0	100.0
Buses	44.1	55.9	100.0	60.4	39.6	100.0
All commercial vehicles	36.0	64.0	100.0	67.1	32.9	100.0

Table 7.—Number of commercial vehicles crossing the border on an average summer weekday in 1954, according to country of home registration, number registered in more than one place, and class of truck operation

Place of registration	Class of truck operation		Buses	Total
	Private	For hire		
United States registered vehicles:				
Registered in United States only	533	256	56	845
Registered in United States and one province	75	138	38	251
Registered in United States and two provinces	-----	8	---	8
Registered in United States and three provinces	2	3	4	9
Total	610	405	98	1,113
Canadian registered vehicles:				
Registered in Canada only	831	229	79	1,139
Registered in Canada and one State	389	284	36	709
Registered in Canada and two States	9	41	4	54
Registered in Canada and three States	19	26	4	49
Registered in Canada and four States	1	17	1	19
Registered in Canada and five States	3	4	---	7
Registered in Canada and six States	1	---	---	1
Registered in Canada and seven States	1	---	---	1
Registered in Canada and nine States	-----	1	---	1
Total	1,254	602	124	1,980

classified according to country of registration and major vehicle types, is shown in table 9.

Trips made no oftener than once a year constituted 11.4 percent of the total. At the other extreme, trips made oftener than once a day amounted to 19.1 percent. More than half of the trips, 56.0 percent, were repeated more often than once a week, on the average. There was no consistent difference in trip frequency with respect to United States and Canadian registered vehicles.

The trips made very frequently were, of course, short trips. One trip was reported to have been made as frequently as 6,000 times per year and was only 1½ miles long—one-half mile of travel in the United States and one mile in Canada. The vehicle was registered in the United States and hauled coal from Canada and returned empty. The very long trips were mostly nonrepetitive or very infrequent.

Trip Length Varies with Commodity Hauled

The commodities hauled by commercial vehicles are classified in tables 10 and 11 according to the 1954 edition of the *Freight Commodity Statistics Classification* published by the Association of American Railroads.

Of the 3,093 vehicles recorded, as shown in table 10, 222 or 7.2 percent were buses, 1,432 or 46.3 percent were empty trucks, and 1,439 or 46.5 percent were loaded trucks. That is, 92.8 percent of the commercial vehicles were trucks and about half were empty. The average length of trip by empty trucks was 155 miles compared with 353 miles for loaded trucks. However, the proportion of travel by empty and loaded trucks in the United States and Canada was about the same, being 66 and 34 percent for empty trucks and 68 and 32 percent for loaded trucks, respectively.

Agricultural products

Trucks hauling agricultural products accounted for 4.5 percent of all commercial vehicles and 4.2 percent of the mileage traveled. Over 61 percent of the mileage traveled was in the United States. The average length of trip was 231 miles: 142 miles in the United States and 89 miles in Canada. The largest movement of agricultural products occurred at Detroit, stations 7–8, with the next heaviest movements at Champlain, N. Y., station 11,

Table 8.—Number of commercial vehicles crossing the border on an average summer weekday in 1954, according to type of vehicle, country of registration, and truck body type

Type of vehicle and country of registration	Truck body type								Other truck body types and buses	All vehicles
	Panel	Tank	Stake or platform	Refrigerator	Van or covered	Dump	Box	Cattle rack		
Single-unit trucks:										
2-axle, 4 tire:										
United States	88	1	20	1	10	-------	155	3	17	295
Canada	225	1	60	-------	16	1	305	-------	18	626
2-axle, 6 tire:										
United States	8	6	116	1	35	81	15	3	24	289
Canada	2	11	262	4	87	43	98	11	8	526
All single-unit trucks:										
United States	96	7	136	2	45	81	170	6	41	584
Canada	227	12	322	4	103	44	403	11	26	1,152
Total	323	19	458	6	148	125	573	17	67	1,736
Truck-tractor and semitrailer combinations:										
3-axle:										
United States	1	5	37	4	65	-------	5	2	59	178
Canada	-------	12	130	3	129	-------	5	3	34	316
4-axle or more:										
United States	-------	8	42	24	120	-------	32	1	7	234
Canada	-------	9	69	49	186	10	10	4	17	354
Truck and trailer combinations:										
United States	-------	5	6	-------	5	1	-------	-------	2	19
Canada	-------	9	10	1	11	-------	1	-------	2	34
All combinations:										
United States	1	18	85	28	190	1	37	3	68	431
Canada	-------	30	209	53	326	10	16	7	53	704
Total	1	48	294	81	516	11	53	10	121	1,135
Buses:										
United States	-------	-------	-------	-------	-------	-------	-------	-------	98	98
Canada	-------	-------	-------	-------	-------	-------	-------	-------	124	124
Total	-------	-------	-------	-------	-------	-------	-------	-------	222	222
All vehicles:										
United States	97	25	221	30	235	82	207	9	207	1,113
Canada	227	42	531	57	429	54	419	18	203	1,980
Total	324	67	752	87	664	136	626	27	410	3,093

nd Sweetgrass, Mont., station 2. About 60 ercent of the movement was from Canada to he United States. More than three-fourths f the trucks moving agricultural products vere registered in Canada and two-thirds of hese vehicles were hauling from Canada to he United States.

nimals and animal products

Approximately the same number and per- entage of trucks were hauling animals or nimal products as were hauling agricultural roducts, namely, 142 trucks or 4.6 percent f all vehicles. However, the mileage traveled y trucks hauling animals and animal prod- cts was quite different—almost three times hat for agricultural products. The average rip length was 650 miles: 456 miles in the Jnited States and 194 miles in Canada. eventy percent of the travel was in the Jnited States.

About two-thirds of the movement of ani- nals and animal products was from Canada to the United States and the major portion of the vehicles were registered in Canada. More trucks hauling animal products were recorded at the Detroit, Buffalo, and Calais, Maine stations than at other stations. The most frequent item at Calais was fish and lobsters.

Table 9.—Frequency of identical trips made by commercial vehicles crossing the border on an average summer weekday in 1954, according to country of registration and type of vehicle

Country of registration and type of vehicle	Frequency of identical trips						
	Trip made no oftener than—					Oftener than twice a day	All trips
	Once a year	Once a month	Once a week	Once a day	Twice a day		
United States registered vehicles:							
Single-unit trucks	88	87	67	200	27	115	584
Truck combinations	54	101	106	146	9	15	431
Buses	16	7	4	65	4	2	98
Total	158	195	177	411	40	132	1, 113
Canadian registered vehicles:							
Single-unit trucks	134	172	210	438	97	101	1, 152
Truck combinations	53	66	176	219	102	88	704
Buses	9	7	4	74	10	20	124
Total	196	245	390	731	209	209	1, 980
All commercial vehicles:							
Single-unit trucks	222	259	277	638	124	216	1, 736
Truck combinations	107	167	282	365	111	103	1, 135
Buses	25	14	8	139	14	22	222
Total	354	440	567	1, 142	249	341	3, 093

able 10.—Number and percentage distribution of commercial vehicles (loaded and empty) crossing the border on an average summer weekday in 1954, according to commodity hauled, mileage traveled, and average length of trip

Commodity or vehicle classification	Vehicles crossing border			Total distance traveled			Average distance traveled in—					
	Number of vehicles	Percent of all vehicles	Percent hauling commodities	Miles	Percent of all mileage	Percent hauling commodities	United States		Canada		Total	
							Miles	Percent	Miles	Percent	Miles	Percent
TRUCKS HAULING COMMODITIES												
Products of agriculture	141	4. 5	9. 8	32, 636	4. 2	6. 4	142	61. 5	89	38. 5	231	100. 0
Animals and animal products	142	4. 6	9. 9	92, 300	11. 9	18. 2	456	70. 2	194	29. 8	650	100. 0
Products of mines	76	2. 5	5. 3	2, 293	. 3	. 4	15	50. 0	15	50. 0	30	100. 0
Products of forests	111	3. 6	7. 7	9, 583	1. 2	1. 9	47	54. 7	39	45. 3	86	100. 0
Manufactured and miscellaneous products	969	31. 3	67. 3	370, 898	47. 7	73. 1	264	68. 9	119	31. 1	383	100. 0
All commodities	1, 439	46. 5	100. 0	507, 710	65. 3	100. 0	241	68. 3	112	31. 7	353	100. 0
BUSES AND EMPTY TRUCKS												
Empty trucks	1, 432	46. 3	-------	222, 046	28. 6	-------	102	65. 8	53	34. 2	155	100. 0
Buses	222	7. 2	-------	47, 155	6. 1	-------	128	60. 4	84	39. 6	212	100. 0
ALL COMMERCIAL VEHICLES												
All commercial vehicles	3, 093	100. 0	-------	776, 911	100. 0	-------	169	67. 3	82	32. 7	251	100. 0

able 11.—Number of commercial vehicles (loaded and empty) crossing the border on an average summer weekday in 1954, according to country of registration, origin and destination of trip, and commodity hauled

Commodity or vehicle classification	Origin and destination of travel of United States registered vehicles					Origin and destination of travel of Canadian registered vehicles					Origin and destination of travel of all vehicles				
	Canada to Canada	Canada to United States	United States to Canada	United States to United States	Total	Canada to Canada	Canada to United States	United States to Canada	United States to United States	Total	Canada to Canada	Canada to United States	United States to Canada	United States to United States	Total
TRUCKS HAULING COMMODITIES															
Products of agriculture	-----	14	18	2	34	4	71	32	-----	107	4	85	50	2	141
Animals and animal products	2	34	19	4	59	14	59	10	-----	83	16	93	29	4	142
Products of mines	-----	39	13	-----	52	-----	18	6	-----	24	-----	57	19	-----	76
Products of forests	-----	13	21	7	41	1	41	28	-----	70	1	54	49	7	111
Manufactured and miscellaneous products	7	89	131	135	362	108	178	319	2	607	115	267	450	137	969
All commodities	9	189	202	148	548	127	367	395	2	891	136	556	597	150	1, 439
BUSES AND EMPTY TRUCKS															
Empty trucks	2	207	204	54	467	47	555	362	1	965	49	762	566	55	1, 432
Buses	4	36	51	7	98	1	78	39	6	124	5	114	90	13	222
ALL COMMERCIAL VEHICLES															
All commercial vehicles	15	432	457	209	1, 113	175	1, 000	796	9	1, 980	190	1, 432	1, 253	218	3, 093

Mine products

The smallest number of trips, 76 or only 2.5 percent of the total, were made by vehicles carrying mine products. Mileage traveled by these vehicles was only 0.3 percent of the total. The trip length was the shortest for any commodity hauled and amounted to only 30 miles, equally divided between the United States and Canada. This is the only product hauled where the portion of travel in Canada equaled that in the United States.

Of the 76 vehicles hauling mine products, 52 were registered in the United States and 39 of these vehicles were moving from Canada to the United States. Nearly half of the vehicles hauling mine products were recorded at the Madawaska, Maine, station.

Forest products

Vehicles transporting forest products accounted for 3.6 percent of all vehicles, but their travel amounted to only 1.2 percent of the total. The average length of trip was 86 miles: 47 miles in the United States and 39 miles in Canada.

The heavier movements in this category occurred at Jackman, Maine, and International Falls, Minn. Almost 50 percent of the vehicles hauling forest products were recorded at these two stations. It is somewhat surprising to find that the movement of forest products from the United States to Canada almost equaled that from Canada to the United States being 44 and 49 percent, respectively. The remaining 7 percent was made up of trips with both origins and destinations in the same country.

Of the five vehicles hauling pulpwood, all were traveling from Canada to the United States and the vehicles were registered in the United States.

Manufactured and miscellaneous products

Almost a third of the total vehicles crossing the border carried manufactured products and accounted for almost one-half (47.7 percent) of the total travel. Two-thirds of all loaded vehicles carried these products. With the exception of animals and animal products, trip lengths of vehicles carrying manufactured products were the longest, 383 miles. Travel was divided as follows: 264 miles in the United States and 119 miles in Canada. Sixty-three percent of the vehicles hauling manufactured products were registered in Canada and 37 percent in the United States. Previously it was noted that there were more trips to the United States than to Canada by trucks hauling products of agriculture, animal products, and products of mines and forests. However, there were more trips to Canada than to the United States by vehicles loaded with manufactured and miscellaneous products.

Nearly half (45 percent) of the vehicles hauling manufactured products crossed the border at Port Huron and Detroit, Mich. A large number of trucks hauling this type of commodity were also recorded at Buffalo, N. Y. In fact, more vehicles hauling manufactured products crossed the border at Buffalo than at any other station except the bridge at Detroit. Seventeen vehicles were transporting gasoline and 16 of these were registered in Canada. All trips but one were from the United States to Canada.

Of the 47 vehicles loaded with manufactured iron and steel items, 35 were registered in Canada. There were 28 such vehicles traveling from the United States to Canada.

Vehicles hauling automobiles, trucks, and parts pertaining to the motor industry totaled 211. Of these, 143 were registered in Canada and 68 in the United States. There were 119 such vehicles traveling from the United States to Canada, 25 from Canada to the United States, 40 with both origins and destinations in the United States, and 27 with both origins and destinations in Canada. New autos and trucks were usually being moved between points in the same country, but a portion of the trip was made in the other country.

Of the 12 vehicles hauling newsprint paper 8 were moving from the United States to Canada, 2 from Canada to the United States, and 2 had origins and destinations within Canada

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New Publication

A new publication entitled *Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways, 1956: a reference guide outline* is available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 55 cents a copy.

This reference guide outline was prepared by the Photogrammetry for Highways Committee with active participation by the Bureau of Public Roads. The Photogrammetry for Highways Committee is jointly sponsored by The American Society of Photogrammetry and The American Congress on Surveying and Mapping.

The purpose of this publication is to aid highway officials in establishing specifications for the procurement by contract of photogrammetric and aerial survey services. Under the provisions of the Federal-Aid Highway Act of 1956, the Secretary of Commerce may authorize the use of photogrammetric methods in mapping, and the utilization of commercial enterprise for such services.

The specifications are a reference guide outline, and should be modified to fulfill special requirements not discussed in detail in this publication.

U. S. GOVERNMENT PRINTING OFFICE: 1956

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- 1950, 25 cents.
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- 1954 (out of print).
- 1955, 25 cents.

PUBLICATIONS

Bibliography of Highway Planning Reports (1950). 30 cents.

Braking Performance of Motor Vehicles (1954). 55 cents.

Construction of Private Driveways, No. 272MP (1937). 15 cents.

Criteria for Prestressed Concrete Bridges (1954). 15 cents.

Design Capacity Charts for Signalized Street and Highway Intersections (reprint from PUBLIC ROADS, Feb. 1951). 25 cents

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Financing of Highways by Counties and Local Rural Governments: 1931–41, 45 cents; 1942–51, 75 cents.

General Location of the National System of Interstate Highways, Including All Additional Routes at Urban Areas Designated in September 1955. 55 cents.

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Highway Statistics (annual):
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- 1946, 50 cents.
- 1947, 45 cents.
- 1948, 65 cents.
- 1949, 55 cents.
- 1950 (out of print).
- 1951, 60 cents.
- 1952, 75 cents.
- 1953, $1.00.
- 1954, 75 cents.

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Specifications for Construction of Roads and Bridges in National Forests and National Parks, FP-41 (1948). $1.50.

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Taxation of Motor Vehicles in 1932. 35 cents.

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Transition Curves for Highways (1940). $1.75.

MAPS

State Transportation Map series (available for 39 States). Uniform sheets 26 by 36 inches, scale 1 inch equals 4 miles. Shows in colors Federal-aid and State highways with surface types, principal connecting roads, railroads, airports, waterways, National and State forests, parks, and other reservations. Prices and number of sheets for each State vary—see Superintendent of Documents price list 53.

United States System of Numbered Highways. 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

Bibliography on Automobile Parking in the United States (1946).

Bibliography on Highway Lighting (1937).

Bibliography on Highway Safety (1938).

Bibliography on Land Acquisition for Public Roads (1947).

Bibliography on Roadside Control (1949).

Express Highways in the United States: a Bibliography (1945).

Indexes to PUBLIC ROADS, volumes 17–19 and 23.

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DEPARTMENT OF COMMERCE—BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF AUG. 31, 1956

State	Unprogramed balances[1]	Active program											
		Programed only			Plans approved, construction not started			Construction under way			Total		
	(Thousand dollars)	Thousand dollars		Miles	Thousand dollars		Miles	Thousand dollars		Miles	Thousand dollars		Miles
		Total cost	Federal funds		Total cost	Federal funds		Total cost	Federal funds		Total cost	Federal funds	
Alabama	$69, 025	$20, 878	$12, 944	312. 0	$3, 837	$2. 050	41. 0	$50, 011	$27, 010	649. 2	$74. 726	$42, 004	1, 002. 2
Arizona	41, 540	6, 127	3, 958	55. 4	3, 655	2, 644	40. 0	11, 419	8, 608	123. 2	21, 201	15, 210	218. 6
Arkansas	57, 840	16, 190	8, 127	506. 9	7, 044	3, 709	56. 3	21, 530	11, 284	388. 2	44, 764	23, 120	951. 4
California	183, 965	46, 445	31, 356	280. 0	9, 868	5, 004	14. 2	142, 142	73, 616	229. 6	198, 455	109, 976	523. 8
Colorado	56, 019	11, 359	6, 964	135. 6	1, 266	464	5. 6	28, 210	16, 950	240. 2	40, 835	24, 378	381. 4
Connecticut	54, 668	2, 876	1, 438	2. 6	990	600	. 8	11, 437	5, 812	24. 2	15, 303	7, 850	27. 6
Delaware	24, 660	1, 260	630	22. 3	3, 264	1, 652	17. 9	5, 936	2, 952	50. 4	10, 460	5, 234	90. 6
Florida	60, 868	18, 844	9, 481	340. 7	8, 622	4, 859	32. 6	42, 063	21, 713	328. 8	69, 529	36, 053	702. 1
Georgia	90, 086	36, 612	19, 050	633. 1	6, 602	3, 135	31. 8	58, 821	28, 638	875. 9	102, 035	50, 823	1, 540. 8
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South Carolina	51, 792	17, 286	9, 480	353. 2	4, 827	2, 604	24. 0	22, 210	12, 062	495. 9	44, 323	24, 146	873. 1
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Vermont	24. 640	774	387	15. 3	308	154	1. 4	11, 016	5, 668	86. 9	12, 098	6, 209	103. 6
Virginia	78, 117	17, 714	9, 483	260. 0	6, 037	3, 211	87. 8	33, 888	17, 271	330. 9	57, 639	29, 965	678. 7
Washington	56, 776	14, 092	8. 847	119. 4	7, 610	4. 338	99. 4	31, 315	16, 970	259. 6	53, 017	30, 155	478. 4
West Virginia	52, 941	11, 999	6, 204	49. 2	6, 249	3, 142	38. 9	18, 083	9, 166	38. 2	36, 331	18, 512	126. 3
Wisconsin	86, 525	18. 285	9, 271	218. 2	5, 958	3, 321	17. 5	48, 428	23, 919	470. 3	72, 671	36, 511	706. 0
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District of Columbia	24, 966	11, 801	6, 931	3. 7	1, 007	496	. 1	9, 211	4, 579	1. 9	22, 019	12, 006	5. 7
Puerto Rico	12, 997	5. 554	2, 259	18. 0	3, 301	1, 515	1. 4	18, 362	8, 538	63. 3	27, 217	12. 312	82. 7
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[1] Includes funds for fiscal year 1958, apportioned Aug. 1, 1956.

VOL. 29, NO. 5

DECEMBER 1956

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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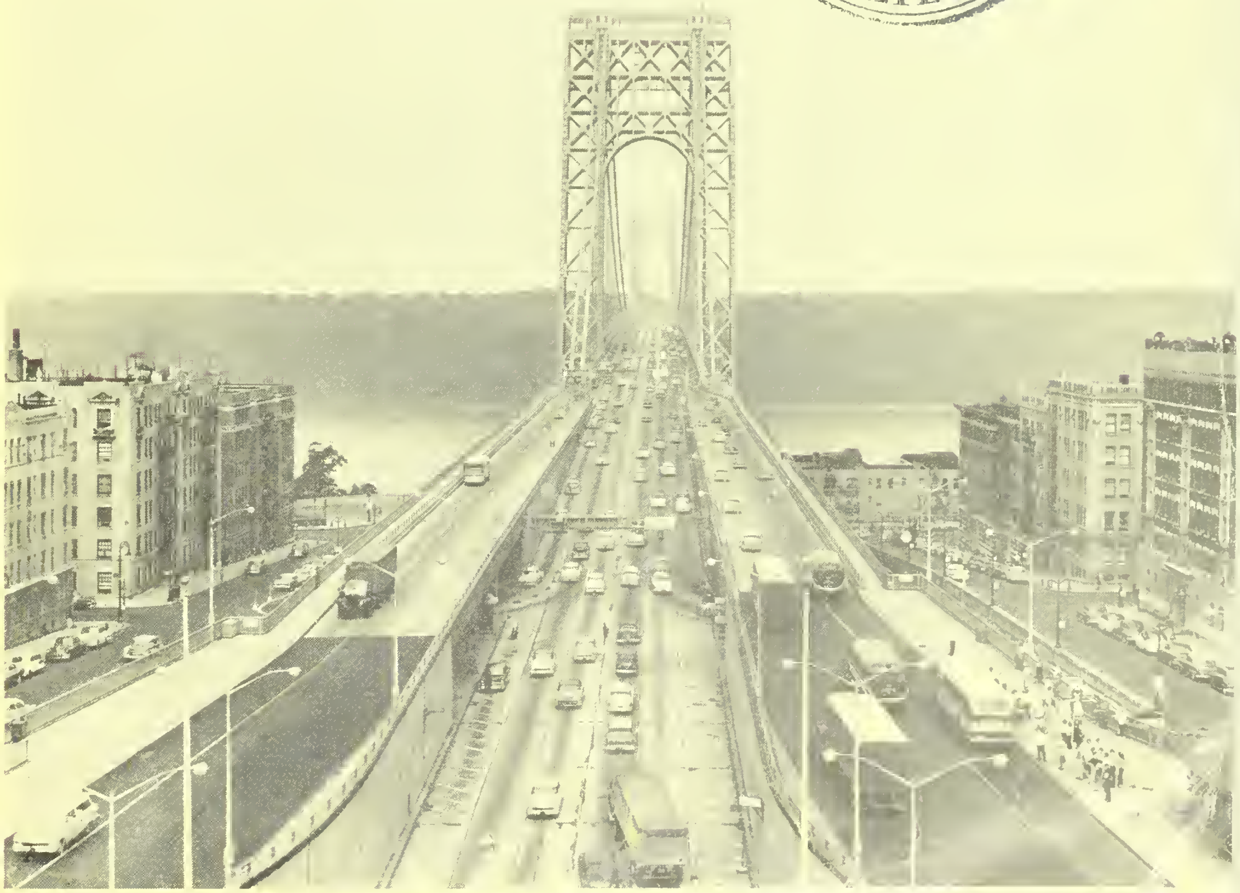

The George Washington Memorial Bridge, New York City.
(An average of 98,000 vehicles used this bridge each day during 1955)

Public Roads

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C. M. Billingsley, *Editor*

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Washington 25, D. C.

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Use of funds for printing this publication approved by the Director of the Bureau of the Budget, Mar. 17, 1955.

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Traffic and Travel Trends, 1955

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by THOMAS B. DIMMICK
Head, Current Data Analysis Unit

The tremendous growth of the highway transportation industry is revealed in the considerable body of travel data presented in this article, which are collected by means of automatic traffic recorders installed at a large number of locations in all States, as well as from a number of loadometer or pitscale stations operated in 44 States for the purpose of studying vehicle types, weights, and loading practices.

From 1950 to 1955, travel on all roads and streets increased approximately 32 percent; travel on rural roads increased 40 percent, whereas urban travel increased almost 23 percent. During this same period, passenger-car travel increased 34 percent; trucks and truck combinations, 23 percent; and buses, 10 percent.

This aticle stresses primarily the travel trends and loading practices on the main rural roads of the country, which comprise about 12 percent of all rural mileage and carry 68 percent of all rural traffic. Approximately two-thirds of all truck travel in rural areas is performed on these roads, of which about one-third is by combination-type vehicles.

Average daily travel on the main rural roads totaled 595 million vehicle-miles in 1954, as compared with 625 million in 1955, a 5.1 percent increase. By principal geographical areas, the rate of increase in travel from 1954 to 1955 was 7.1 percent for the States in the eastern divisions, 5.4 percent for States in the western divisions, and 3.6 percent for States in the central divisions. The greatest increase in travel took place in States of the Middle Atlantic division, and was followed by the South Atlantic division.

Passenger-car and bus travel on main rural roads increased 147 percent between 1936 and 1955; 103 percent between 1940 and 1955; and 34 percent between 1950 and 1955. For single-unit trucks, the increases were 154 percent, 93 percent, and 18 percent; whereas truck combinations increased 455 percent, 248 percent, and 22 percent, respectively.

A special survey undertaken during the summer of 1955 indicated that 73 percent of the truck travel on main rural roads was performed by private haulers, and the remaining 27 percent was by for-hire carriers of which 18 percent had ICC authority numbers. Of the total truck travel on main rural roads, approximately 30 percent involved trips in more than one State; the remaining 70 percent were intrastate trips made largely by private haulers.

In 1955, 55 percent of all freight-carrying vehicles were loaded, and weighed an average of 24,336 pounds. The weight of empty vehicles averaged 9,426 pounds. For the period 1950–55, weights of loaded single-unit trucks increased 3 percent, whereas combinations increased over 6 percent.

Single-unit trucks in 1955 carried loads during 48 percent of their travel as compared with 60–65 percent during the prewar period 1936–41. Combinations in 1955 were found to be loaded during 68 percent of their travel as compared with 72 percent in 1936.

Average loads carried by single-unit trucks increased from 1.86 tons in 1936 to 2.47 tons in 1955 (33 percent increase), while combinations increased from 6.90 tons in 1936 to 11.07 tons in 1955 (60 percent increase).

Ton-mileage hauled in 1936 by single-unit trucks was 14.3 billion as compared with 38.5 billion in 1955; combinations in 1936 hauled 13.7 billion ton-miles as compared with 115.6 billion in 1955. The two-axle, six-tire trucks, the principal load-carrying single-unit trucks, accounted for 26 percent of all truck travel in 1955, and 17 percent of the ton-mileage hauled; truck-tractor and semitrailer combinations accounted for slightly less than 30 percent of the travel, but carried nearly 68 percent of the ton-mileage.

Frequencies of freight-carrying vehicles weighing 30,000, 40,000, and 50,000 pounds or more reached a new high in 1955. Since 1936 the number of trucks in each 1,000 loaded and empty vehicles weighing 30,000 pounds or more have increased almost 5 times; for 40,000 pounds or more, over 11 times; and 50,000 pounds or more, 25 times. From 1950 to 1955, the frequencies increased 10, 16, and 29 percent, respectively.

The frequencies of axles weighing 18,000, 20,000, and 22,000 pounds or more show an increase in 1955 over 1954, but for the period 1950–55, there has been a decrease of 9, 20, and 35 percent in the three respective axle-weight categories.

UNDER the greatly expanded highway program authorized by the Federal-Aid Highway Act of 1956, it is readily apparent that measurement of highway usage through traffic-counting and sampling procedures will play a significant role in the planning and design of traffic facilities. The huge program of completing within 13–15 years a 41,000-mile network of modern roads on the National System of Interstate and Defense Highways requires that traffic volumes must be determined as far in advance as 1975, in order that these highways be so designed that they will not become functionally obsolete before the surfaces wear out. Such future traffic demands can be estimated only on the basis of trends developed in the past under conditions that can be expected to be similar to those that will most likely be encountered in the future.

In the 5-year period beginning in 1936, 47 of the 48 States, in cooperation with the Bureau of Public Roads, conducted surveys for a 12-month period to collect data which would supply comprehensive information concerning vehicle characteristics and travel habits. The measuring of road mileages, the counting of traffic and classification by vehicle type, the weighing of trucks on rural roads, and the questioning of drivers concerning origin and destination of trips and mileage driven on different road systems during the preceding 12 months, supplied basic data from which a vast amount of information regarding travel habits, ton-miles hauled on rural highway systems, and vehicle-miles driven on all road systems could be determined for the period of the survey.

Shortly after the original surveys were completed, the States installed automatic traffic recorders at a large number of locations and adopted other continuing operations to provide data for estimating trends in traffic volumes. Such recorders were usually located on main rural roads, but in some instances they were located on the local roads and on city streets. Periodic weighing operations, combined with manual counts of all vehicles passing the weighing stations, have been made which provide information concerning vehicle types and weights as well as their loading habits. By means of these trends, annual estimates have been published, showing for each year the travel on rural roads and city streets from 1936 to 1954.[1] By combining carried load data with vehicle-mileage figures, the ton-mileage of freight hauled on main rural roads has been estimated for each year. Sufficient

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[1] See previous articles on traffic in PUBLIC ROADS: vol. 28, No. 11; vol. 27, Nos. 6 and 11; vol. 26, Nos. 5 and 11; vol. 25, Nos. 3, 7, and 12; vol. 24, No. 10; and vol. 23, No. 9.

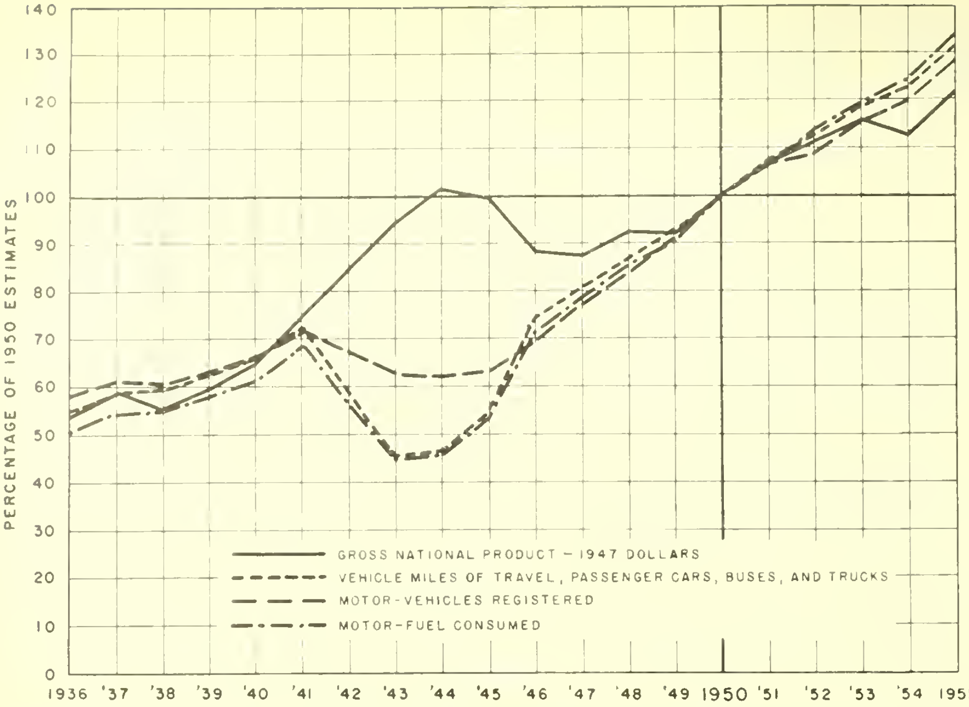

Figure 1.—Total travel, motor-vehicle registration, motor-fuel consumption, and Gross National Product, 1936–55, as a percentage of the respective amounts in 1950.

data are not available to permit the estimating of ton-mileage carried on local roads within reasonable limits of accuracy. No data are available concerning loads carried on city streets and no attempts have been made to estimate the amounts of this haulage.

Urban Travel Data Improve

In recent years many States have adopted some plan for observing trends in urban travel either by means of continuous or part-time traffic counts, and several States have made sufficient counts to enable them to estimate accurately the travel on streets of their key cities. Most States are now able to report the average daily travel on the urban extensions of State highway systems, and other arterial streets. While much is desired in the way of more complete urban travel data for these major thoroughfares, considerable progress is being made.

Although approximations may have to be made concerning the lightly traveled local streets, a large expenditure of funds to determine accurately the amount of travel on them is not always justifiable. Motor-vehicle-use surveys made in several States have provided some additional data concerning all urban travel.

One important result of State cooperation in the study of needs of the highway systems, required by the Federal-Aid Highway Act of 1954, was a complete estimate of vehicle-miles of travel in the previous year on all systems, urban as well as rural. These State estimates for 1953 made it possible to establish a new base period for estimating future traffic trends.

Table 1.—Estimate of motor-vehicle travel in the United States by vehicle types in the calendar year 1955

Vehicle type	Motor-vehicle travel					Number of registered vehicles[1]	Average travel per vehicle	Motor-fuel consumption		Average travel per gallon of fuel consumed
	Main rural road travel	Local rural road travel	Total rural travel	Urban travel	Total travel			Total[2]	Average per vehicle	
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles/gal.*
Passenger cars[3]	178,732	84,356	263,088	224,452	487,540	52,092	9,359	33,548	644	14.53
Buses:										
Commercial	1,152	300	1,452	1,804	3,256	96	34,035	651	6,807	5.00
School and nonrevenue	623	504	1,127	124	1,251	159	7,836	120	750	10.45
All buses	1,775	804	2,579	1,928	4,507	255	17,658	771	3,021	5.85
All passenger vehicles	180,507	85,160	265,667	226,380	492,047	52,347	9,400	34,319	656	14.34
Trucks and combinations	47,481	23,005	70,486	40,901	111,387	10,413	10,697	13,308	1,278	8.37
All vehicles	227,988	108,165	336,153	267,281	603,434	62,760	9,615	47,627	759	12.67

[1] Registration figures differ slightly from those in Bureau of Public Roads table MV-1 because of adjustments in classification in a few States of lightweight farm trucks.

[2] Total fuel consumed differs from that given in Bureau of Public Roads table G-21 because of adjustments to cover estimated amounts used by motorcycles.

[3] Includes taxicabs.

Traffic and Gross National Product

The travel on all rural roads and streets, motor-vehicle registrations, motor-fuel consumption, and the Gross National Product (in constant dollars) are shown in figure 1 for the years 1936–55, inclusive, as a percentage of the 1950 totals. This chart indicates that with the exception of the war years and a few years thereafter when traffic restrictions drastically curtailed travel, and while production was stimulated, the trend of total travel follows closely the economic trend as represented by the Gross National Product. In 1954 the Gross National Product decreased slightly but there was a lesser degree of leveling off in traffic, whereas in 1955, there was a parallel trend. The indexes of motor-vehicle registrations and motor-fuel consumption follow the same trend, which indicates that driving habits of the average car owner remain fairly constant in normal years.

Table 1 shows the estimated amounts of travel in 1955 on main rural roads, local rural roads, and urban streets for passenger cars, buses, and trucks together with the number of vehicles registered and quantity of motor fuel consumed on highways. The travel figures were obtained mainly by applying the available trends to the 1953 data which, as previously stated, were derived from the various State reports submitted for the nationwide highway study. Actual vehicle-mileage totals on the various turnpikes were obtained from reports of the turnpike authorities. Minor adjustments were made in the totals of local rural road and city street travel to include the generated or induced travel on improved or newly constructed sections of lightly traveled feeder roads and on streets in expanding fringe areas where traffic counts ordinarily are not made.

In addition, table 1 shows average miles of travel per vehicle, average consumption of motor fuel, and average travel per gallon of fuel consumed. These figures are included merely as checks on the reasonableness of the vehicle-mileage estimates when compared with other factual data.

Travel Continues Upward Trend

Available data indicate the following increases in travel from 1950 to 1955: rural areas, 40 percent; urban areas, almost 23 percent; and total travel, about 32 percent. Registrations have increased 28 percent and the quantity of motor fuel used on the highways has increased 34 percent. The miles traveled per vehicle and fuel consumed per vehicle increased about 3 and 4 percent, respectively, whereas the miles traveled per gallon of fuel consumed decreased about 2 percent.

A comparison of travel on all roads and streets in 1940, 1945, 1950, and 1955 is given in table 2. Probably the most significant relation is the greater long-range increase in travel by trucks and truck combinations in comparison with passenger cars and buses. This increase is particularly noticeable between 1940 and 1950. Travel by trucks and truck combinations in 1955 was more than

Table 2.—Comparison of estimated vehicle-miles of travel on all roads and streets in 1940, 1945, 1950, and 1955

Year	All vehicles, vehicle-miles	Passenger cars		Buses		Trucks and combinations	
		Percentage of all vehicles	Vehicle-miles	Percentage of all vehicles	Vehicle-miles	Percentage of all vehicles	Vehicle-miles
	Millions		*Millions*		*Millions*		*Millions*
1940	302,188	82.60	249,604	0.88	2,657	16.52	49,927
1945	250,173	80.02	200,199	1.53	3,832	18.45	46,142
1945: 1940 ratio	*.83*	*.97*	*.80*	*1.74*	*1.44*	*1.12*	*.92*
1950	458,246	79.35	363,613	.89	4,081	19.76	90,552
1950: 1945 ratio	*1.83*	*.99*	*1.82*	*.58*	*1.06*	*1.07*	*1.96*
1950: 1940 ratio	*1.52*	*.96*	*1.46*	*1.01*	*1.54*	*1.20*	*1.81*
1955	603,434	80.79	487,540	.75	4,507	18.46	111,387
1955: 1950 ratio	*1.32*	*1.02*	*1.34*	*.84*	*1.10*	*.93*	*1.23*
1955: 1945 ratio	*2.41*	*1.01*	*2.44*	*.49*	*1.18*	*1.00*	*2.41*
1955: 1940 ratio	*2.00*	*.98*	*1.95*	*.85*	*1.70*	*1.12*	*2.23*

twice the amount in 1940. The number of trucks and truck combinations in relation to all other vehicles increased from 16.52 percent in 1940 to 18.46 percent in 1955. Passenger-car travel during the period increased 95 percent. Bus travel leveled off considerably and only a 70-percent gain is indicated for this period. A more detailed study of the growth of bus travel shows that in the period 1940–55, commercial bus travel increased only 67 percent while travel by school and other nonrevenue buses increased 75 percent.

The largest portion of rural travel by trucks and combinations was found on the main roads. These main roads, which comprise only 12 percent of the mileage of all rural roads, actually carry over 67 percent of the truck travel. As would be expected, the larger portion of the heavier vehicles use the main rural roads. Approximately 32 percent of the truck traffic on these roads was made up of combination-type vehicles, but on the local rural roads these combinations accounted for only 10 percent of the traffic. To illustrate further the relation of truck traffic on the two classes of rural roads, the average daily number of trucks per mile of main rural roads in 1955 was 236 single-unit trucks and 112 truck combinations; on the local roads there were 21 and 2 vehicles, respectively.

Data have been collected concerning the loads carried on local roads, but such studies were limited in scope and therefore not as reliable as information on main roads. Local road mileage far exceeds that of the main roads, yet estimates indicate that truck travel on main roads was more than double and ton-mileage hauled was about four times the amount carried on local roads. Because of the limited data and the relative unimportance of the local road mileage from a freight-carrying standpoint, discussion in subsequent sections of this article is confined to the main rural roads.

Figure 2 shows the annual vehicle-miles of travel on main rural roads by 12-month periods ending each month (moving average) from the end of 1936, the first year of the planning surveys, to August 1956. This method of presentation reduces the seasonal fluctuations. The portion of the curve from the end of 1946 through 1950 indicates that the increase averaged over 10 percent each year. Since 1950 the annual increases have been somewhat smaller: 1951, about 9 percent; 1952, 7 percent; 1953, 5 percent; and 1954, 2 percent. Travel on the main rural roads increased at a slightly faster rate in 1955, being more than 5 percent above the 1954 total.

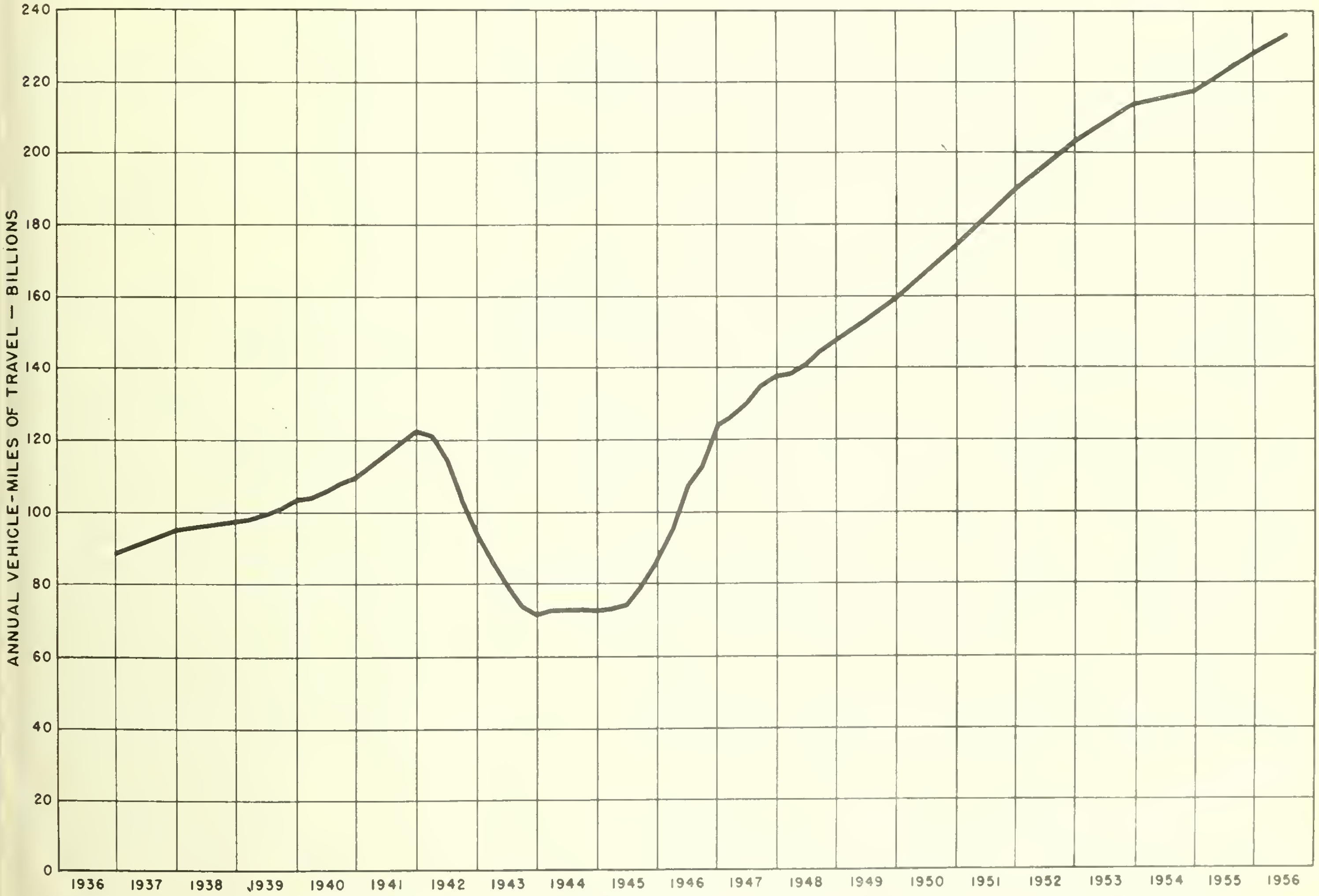

Figure 2.—Vehicle-miles of travel on main rural roads by 12-month periods ending each month, 1936 to August 1956.

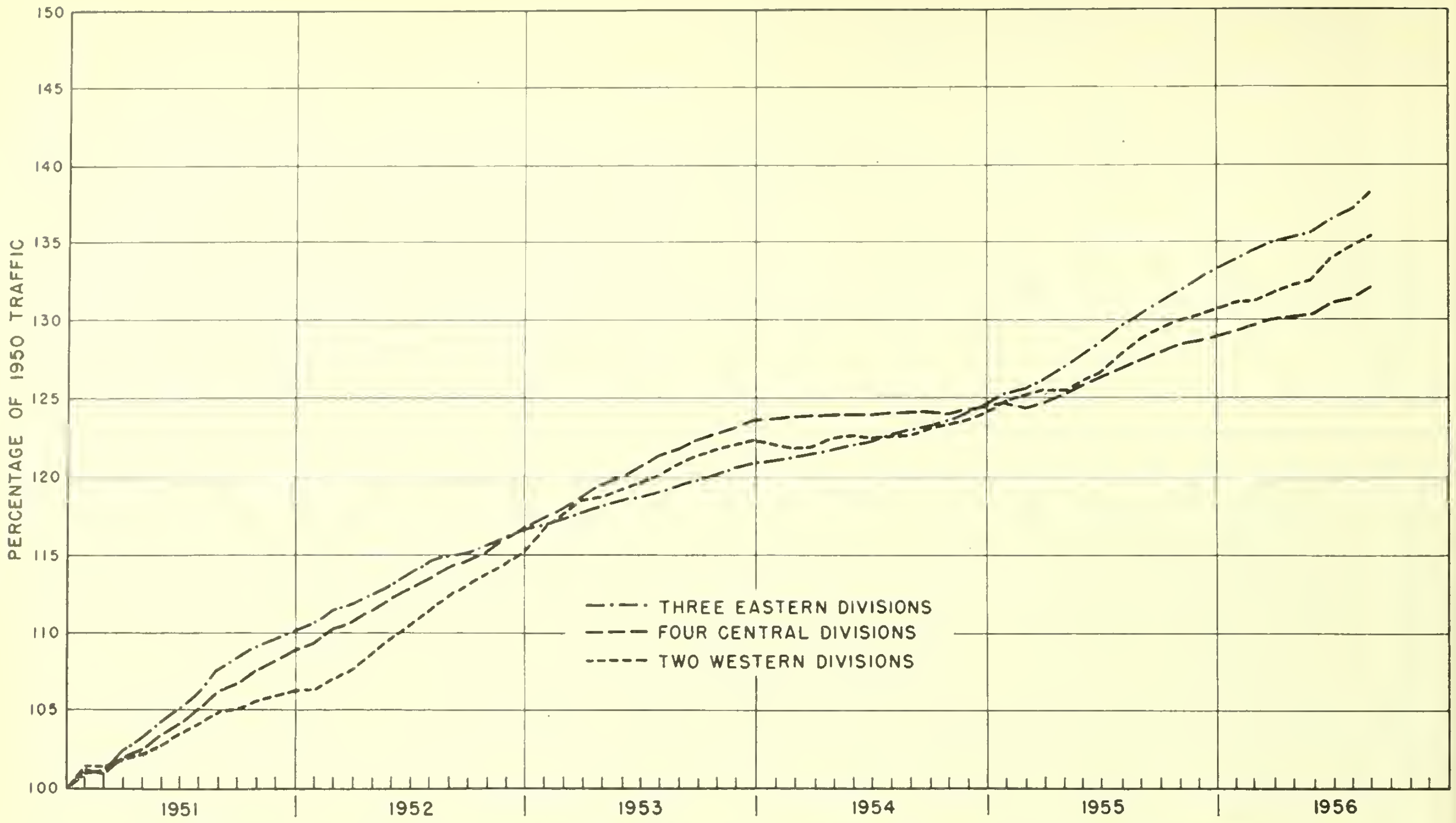

Figure 3.—Travel on main rural roads by 12-month periods ending each month, 1951 to August 1956, as a percentage of travel in 1950.

Geographical Distribution of Travel

Travel on main rural roads in the eastern, central, and western census divisions of the country,[2] by 12-month periods ending each month, is shown in figure 3 as a percentage of such travel in 1950. Since 1950, traffic has consistently increased in all divisions but at a higher rate in the States included in the eastern divisions. During this period, travel in the central divisions likewise increased steadily but at a lower rate, and that in the western divisions increased at a rate somewhere between the rate found in the eastern divisions and that in the central divisions. Increases in the western and central divisions had been greater in the years prior to 1950.

Figure 3 shows that at the end of 1955, travel on main rural roads of the eastern divisions was 33 percent higher than at the end of 1950, whereas in the central divisions it was 29 percent higher, and in the western divisions, it was 31 percent above the 1950 level.

The average daily vehicle-miles of travel on main rural roads by months are shown graphically in figure 4 for 1954, 1955, and the first eight months of 1956. The graph shows that travel in 1955 was generally greater month by month in all divisions than it was in the previous year, except in the central divisions during February and in the western divisions during April. Likewise for the first eight months of 1956, travel exceeded that in 1955 except in the western and central divisions during February.

The average daily travel on main rural roads in 1954 totaled 595 million vehicle-miles as compared with 625 million vehicle-miles in 1955, a 5.1 percent increase. A comparison of average daily travel by divisions in 1954 and 1955 is as follows: Eastern divisions, 195 and 209 million vehicle-miles; central divisions, 298 and 308 million vehicle-miles; and western divisions, 102 and 108 million vehicle-miles, respectively. Percentage increases in 1955 over 1954 for the respective divisions were 7.1, 3.6, and 5.4.

Travel by passenger cars, single-unit trucks, and truck combinations has increased steadily each year since 1950 with the exception of 1954 when travel by both classes of trucks was slightly under the 1953 totals. The increase in travel by passenger cars has averaged 6.8 percent a year during the past 5 years, while the increase in travel by single-unit trucks has increased an average of only 3.6 percent and that of the truck combinations, 4.4 percent.

These increases, although showing a steady gain, are much smaller than those found in some of the previous years. In the period 1945–50, for instance, passenger-car travel increased an average of 20 percent each year, while single-unit truck travel increased almost as much and truck combinations averaged more than 27 percent. The 1950 surveys indicated a gain of almost 33 percent over the previous year for combination vehicles, while the single-unit trucks increased 12 percent. For the entire period from 1936 to 1955, inclusive, the total annual travel by passenger cars increased 147 percent; single-unit trucks, 154 percent; and truck combinations, 455 percent.

Table 3 provides a comparison of the estimated vehicle-miles of travel on main rural roads in 1940, 1945, 1950, and 1955, and the percentage distribution of this travel by main vehicle types. The table shows that total travel in 1955 was 108 percent greater than it was in 1940, that passenger-car and bus travel was 103 percent greater, while truck and truck combination travel had increased 125 percent. Single-unit truck travel increased 93 percent, while truck combination travel increased 248 percent. Other ratios show greater increases for combinations than for single-unit trucks in each instance. Although the 1954 vehicle-mileage totals for both single-unit trucks and truck combinations were slightly below 1953, that reversal of the general trend has now been eliminated and single-unit truck travel in 1955 was up 3.6 percent and truck combination travel was 5.7 percent higher than in the previous year.

Growth of Automobile Travel Exceeds Trucks, 1950–55

The ratios of 1955 traffic on main rural roads to corresponding traffic in 1950 by type of vehicle and by geographical divisions are

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[2] *Eastern divisions*—New England: Connecticut, Maine, Massachusetts, New Hampshire, Rhode Island, and Vermont. Middle Atlantic: New Jersey, New York, and Pennsylvania. South Atlantic: Delaware, Florida, Georgia, Maryland, North Carolina, South Carolina, Virginia, and West Virginia. *Central divisions*—East North Central: Illinois, Indiana, Michigan, Ohio, and Wisconsin. East South Central: Alabama, Kentucky, Mississippi, and Tennessee. West North Central: Iowa, Kansas, Minnesota, Missouri, Nebraska, North Dakota and South Dakota. West South Central: Arkansas, Louisiana, Oklahoma, and Texas. *Western divisions*—Mountain: Arizona, Colorado, Idaho, Montana, Nevada, New Mexico, Utah, and Wyoming. Pacific: California, Oregon, and Washington.

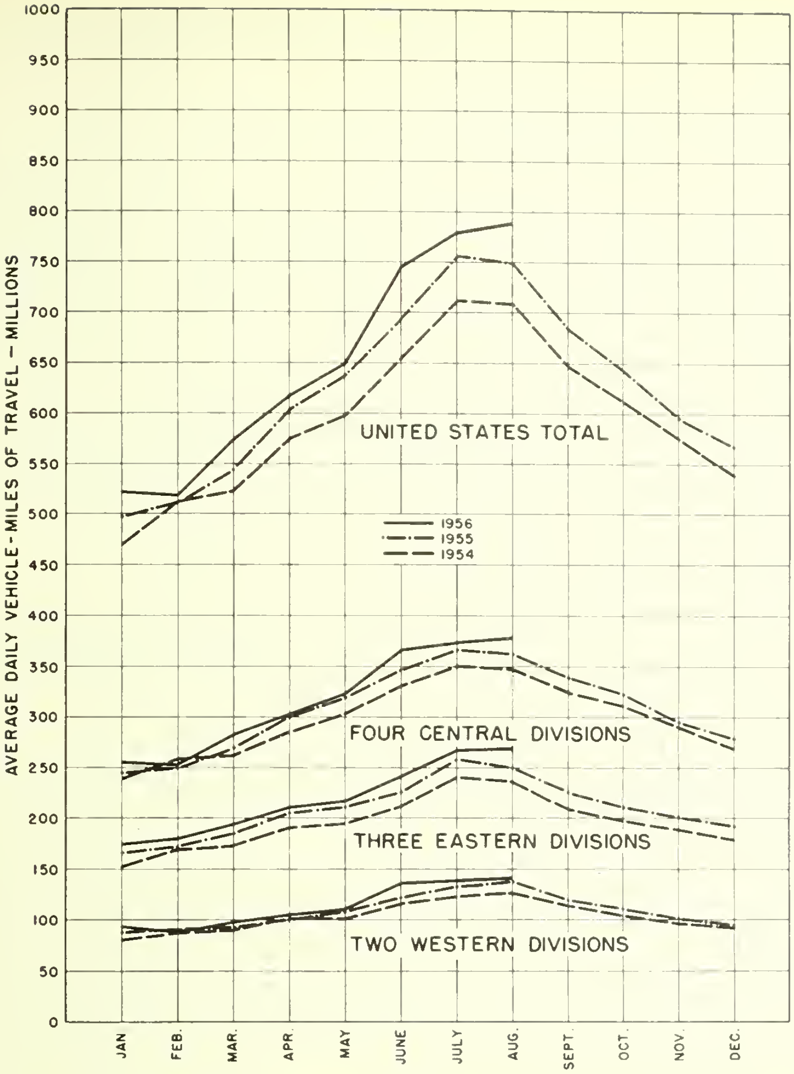

Figure 4.—Average daily travel on main rural roads in 1954, 1955, and in the first 8 months of 1956.

given in table 4. For the United States as a whole, travel increased about 31 percent in 1955 compared with 1950, an average annual increase of 6.2 percent. During this period, passenger-car travel increased 34 percent, an average of 6.8 percent a year; trucks and truck combinations increased 19 percent, 3.8 percent annually; and bus travel decreased 13 percent, 2.6 percent annually. Of the passenger-car travel, that of the local vehicles increased almost 34 percent, while the out-of-State vehicles increased at a slightly faster rate. Travel by the lighter weight single-unit trucks increased 18 percent during the 5-year period or 3.6 percent annually, while travel by the heavier combinations increased 22 percent, an average of 4.4 percent annually. The reduction in bus travel was mostly in the commercial-type vehicles, inasmuch as the travel by school or nonrevenue buses increased about 2.4 percent each year.

The highest percentage increase in traffic was found in the East South Central division with the second highest increase in the South Atlantic division. On the other hand, the lowest percentage increase was found in the Middle Atlantic division with the second lowest rate of increase in the West North Central division.

A comparison of 1955 travel figures with those of the previous year reveals that passenger-car travel increased slightly more than 5 percent; truck and truck combination travel, slightly more than 4 percent; and all travel, 5 percent. The highest rate of increase was found in the Middle Atlantic division, with the second highest rate in the South Atlantic. The large increase in travel found in the Middle Atlantic division was due, to a great extent, to an unusually large number of out-of-State vehicles traveling in that one division.

The percentage distribution of travel by vehicle types on main rural roads in the summer of 1955, according to geographical divisions, is given in table 5. The table shows that the largest percentage of passenger-car travel was in the Pacific division, with that in the New England division following closely. A comparison of truck travel shows that the West South Central division has the highest percentage of travel by all types of trucks and combinations, with the East South Central and Mountain divisions following in order. The lowest percentage for truck travel was found in the Pacific division with the New England division only slightly higher. Travel by all types of combinations in the East North Central division exceeded all others and was followed closely by the West South Central division. When the comparison is restricted to truck and trailer combinations, it can readily be seen that travel by this type is heavily concentrated in the Pacific, Mountain, and East North Central divisions.

When the percentage distribution of travel by types of motor vehicles given in table 5 is compared with a similar distribution previously published for 1950, it is found that during the 1950–55 period, the percentage of passenger-car travel to total travel increased from 76.15 to 78.50; that of single-unit trucks decreased from 15.63 to 14.11 percent; and combinations decreased from 7.21 to 6.72 percent. These figures show, as was noted in the discussion concerning table 4, that travel by trucks and combinations has not expanded as fast as passenger-car travel since 1950.

Special Survey Made in 1955

Certain special information was gathered in the summer of 1955 concerning travel on main rural roads by freight-carrying vehicles. These data provided interesting and useful information concerning freight movements that ordinarily are not available. The information collected made possible the calculation of the percentage relation of travel by single-unit trucks and truck combinations; class of

Table 3.—Comparison of estimated vehicle-miles of travel on main rural roads in 1940, 1945, 1950, and 1955

Year	All vehicles, vehicle-miles	Passenger cars and buses		All trucks and truck combinations		Single-unit trucks		Truck combinations	
		Percentage of all vehicles	Vehicle-miles	Percentage of all vehicles	Vehicle-miles	Percentage of all trucks and truck combinations	Vehicle-miles	Percentage of all trucks and truck combinations	Vehicle-miles
	Millions		*Millions*		*Millions*		*Millions*		*Millions*
1940	109, 815	80. 8	88, 715	19. 2	21, 100	79. 1	16, 699	20. 9	4, 401
1945	85, 792	78. 0	66, 885	22. 0	18, 907	71. 9	13, 602	28. 1	5, 305
1945: 1940 ratio	*.78*	*.97*	*.75*	*1.15*	*.90*	*.91*	*.81*	*1.34*	*1.21*
1950	174, 349	77. 2	134, 527	22. 8	39, 822	68. 4	27, 257	31. 6	12, 565
1950: 1945 ratio	*2.03*	*.99*	*2.01*	*1.04*	*2.11*	*.95*	*2.00*	*1.12*	*2.37*
1950: 1940 ratio	*1.59*	*.96*	*1.52*	*1.19*	*1.90*	*.86*	*1.63*	*1.51*	*2.86*
1955	227, 988	79. 2	180, 507	20. 8	47, 481	67. 8	32, 173	32. 2	15, 308
1955: 1950 ratio	*1.31*	*1.03*	*1.34*	*.91*	*1.19*	*.99*	*1.18*	*1.02*	*1.22*
1955: 1945 ratio	*2.66*	*1.02*	*2.70*	*.95*	*2.51*	*.94*	*2.37*	*1.15*	*2.89*
1955: 1940 ratio	*2.08*	*.98*	*2.03*	*1.08*	*2.25*	*.86*	*1.93*	*1.54*	*3.48*

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Table 4.—Ratio of 1955 traffic on main rural roads to corresponding traffic in 1950, by divisions

Vehicle type	Eastern divisions				Central divisions					Western divisions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Moun-tain	Pacific	Average	
Passenger cars:													
Local	1.61	1.12	1.50	1.35	1.28	1.73	1.19	1.32	1.32	1.29	1.40	1.37	1.34
Foreign	1.11	1.25	1.70	1.43	1.27	1.58	1.32	1.28	1.33	1.31	1.09	1.23	1.35
All passenger cars	1.46	1.15	1.55	1.37	1.28	1.68	1.21	1.31	1.32	1.30	1.36	1.34	1.34
Trucks and truck combinations:													
Single-unit trucks	1.24	1.20	1.34	1.27	1.09	1.11	1.03	1.26	1.13	1.38	1.16	1.26	1.18
Truck combinations	1.13	1.03	1.13	1.09	1.20	1.48	1.32	1.57	1.34	1.29	1.09	1.16	1.22
All trucks and combinations	1.21	1.15	1.28	1.20	1.13	1.19	1.12	1.32	1.17	1.35	1.13	1.22	1.19
Buses	.88	.87	.96	.92	.76	.80	.70	.97	.81	.70	1.14	.96	.87
All vehicles	1.40	1.15	1.48	1.33	1.24	1.51	1.19	1.32	1.29	1.31	1.32	1.32	1.31

Table 5.—Percentage distribution of travel by vehicle types on main rural roads in the summer of 1955, by divisions

Vehicle type	Eastern divisions				Central divisions					Western divisions			United States average
	New England	Middle Atlantic	South Atlantic	Average	East North Central	East South Central	West North Central	West South Central	Average	Moun-tain	Pacific	Average	
Passenger cars:													
Local	63.57	60.61	57.32	59.39	56.49	50.22	62.45	58.73	57.28	43.08	75.41	63.31	59.02
Foreign	19.33	17.70	21.62	19.90	23.17	23.83	17.34	14.47	19.86	32.68	8.53	17.57	19.48
All passenger cars	82.90	78.31	78.94	79.29	79.66	74.05	79.79	73.20	77.14	75.76	83.94	80.88	78.50
Single-unit trucks:													
Panel and pickup	3.87	5.98	7.40	6.38	5.15	9.79	6.22	11.31	7.69	10.64	6.95	8.33	7.37
Other 2-axle, 4-tire	1.48	3.86	.76	1.96	.17	.03	.40	.05	.17	.91	.77	.82	.88
Other 2-axle, 6-tire	5.87	4.99	5.69	5.47	5.18	8.02	5.82	5.69	5.91	6.13	1.91	3.49	5.34
3-axle	.40	.63	.76	.66	.45	.57	.42	.26	.41	.57	.57	.57	.52
All single-unit trucks	11.62	15.46	14.61	14.47	10.95	18.41	12.86	17.31	14.18	18.25	10.20	13.21	14.11
Truck tractor and semitrailer combinations:													
3-axle	3.53	4.31	2.27	3.17	3.16	4.16	2.21	3.15	3.12	1.41	.67	.95	2.77
4-axle	1.15	1.18	3.32	2.25	4.31	2.42	3.40	5.22	4.04	1.29	.59	.85	2.89
5-axle or more	.02	.01	.02	.02	.76	.07	1.00	.38	.60	1.85	1.30	1.50	.56
All truck-tractor and semitrailer combinations	4.70	5.50	5.61	5.44	8.23	6.65	6.61	8.75	7.76	4.55	2.56	3.30	6.22
Truck and trailer combinations:													
4-axle or less	.01	.13	.03	.06	.09	.01	.20	.03	.09	.22	.13	.16	.10
5-axle	--------	.01	.01	.01	.48	--------	.04	--------	.19	.57	1.69	1.27	.31
6-axle or more	--------	--------	--------	--------	.11	--------	.01	--------	.04	.05	.63	.42	.09
All truck and trailer combinations	.01	.14	.04	.07	.68	.01	.25	.03	.32	.84	2.45	1.85	.50
All combinations	4.71	5.64	5.65	5.51	8.91	6.66	6.86	8.78	8.08	5.39	5.01	5.15	6.72
All trucks and truck combinations	16.33	21.10	20.26	19.98	19.86	25.07	19.72	26.09	22.26	23.64	15.21	18.36	20.83
Buses	.77	.59	.80	.73	.48	.88	.49	.71	.60	.60	.85	.76	.67
All vehicles	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00

carrier (Interstate Commerce Commission authority, other for-hire, and private haulers); area of travel (interstate or intrastate trips); and road system traveled (Interstate system or other main roads). In addition, the average trip length of all vehicles sampled was tabulated according to major vehicle types and class of carrier. Since the samples were taken during the summer months, they represent the distribution of traffic in that season only and may not, in some respects, represent the annual distribution.

Table 6 shows the percentage of vehicle-miles of travel by trucks and truck combinations, according to class of carrier and type of trip (interstate or intrastate) in the summer of 1955. This table indicates, as might have been assumed, that the largest portion of travel was by private haulers making intrastate trips on main rural roads other than the Interstate system. The smallest amount of travel was performed by for-hire vehicles operating without ICC authority, and making interstate trips. About 73 percent of all truck travel was by private haulers, and the remaining 27 percent was by for-hire vehicles of which about 18 percent had ICC authority numbers. Slightly over 30 percent of the trips involved interstate travel, and the remainder had origins and destinations within a single State.

In a similar manner table 7 shows the percentage of ton-miles of freight hauled by trucks and truck combinations, according to class of carrier, and type of trip, in the summer of 1955. It is evident from a comparison of tables 6–7 that the heavier loads were carried generally on the Interstate system by vehicles operating with ICC authority numbers and especially, those vehicles engaged in interstate travel. Although vehicles traveling on the Interstate system with ICC authority numbers and making trips through more than one State accounted for 3.42 percent of the total main rural road travel, these vehicles carried 9.15 percent of all freight transported, a ratio of 1 to 2.68. On the other hand, private haulers using the main roads (other than Interstate system) and traveling within a single State accounted for about 45 percent of the travel but only 22 percent of the ton-mileage, or a ratio of 1 to 0.49. Common carriers frequently are able to obtain two-way loads whereas private haulers usually are limited to one-way loads, which may account for the larger proportion of freight transported by the common carriers.

Table 6.—Percentage of vehicle-miles of travel by trucks and truck combinations, by type of carrier and type of trip (interstate or intrastate), in the summer of 1955

System and trip type	ICC authority vehicles	Other for-hire vehicles	Private haulers	All vehicles
Interstate (rural) highway system:				
Interstate trips	3.42	0.62	4.28	8.32
Intrastate trips	1.70	1.25	11.83	14.78
Total	5.12	1.87	16.11	23.10
Other main rural roads:				
Interstate trips	8.49	1.83	11.69	22.01
Intrastate trips	4.77	4.72	45.40	54.89
Total	13.26	6.55	57.09	76.90
All main rural roads:				
Interstate trips	11.91	2.45	15.97	30.33
Intrastate trips	6.47	5.97	57.23	69.67
Total	18.38	8.42	73.20	100.00

Table 7.—Percentage of ton-miles of freight hauled by trucks and truck combinations, by type of carrier and type of trip (interstate and intrastate), in the summer of 1955

System and trip type	ICC authority vehicles	Other for-hire vehicles	Private haulers	All vehicles
Interstate (rural) highway system:				
Interstate trip	9. 15	1. 46	5. 28	15. 89
Intrastate trip	2. 78	2. 37	6. 56	11. 71
Total	11. 93	3. 83	11. 84	27. 60
Other main rural roads:				
Interstate trips	20. 07	3. 09	12. 35	35. 51
Intrastate trips	7. 14	7. 36	22. 39	36. 89
Total	27. 21	10. 45	34. 74	72. 40
All main rural roads:				
Interstate trips	29. 22	4. 55	17. 63	51. 40
Intrastate trips	9. 92	9. 73	28. 95	48. 60
Total	39. 14	14. 28	46. 58	100. 00

Table 8.—Average trip length (one way) of freight-carrying vehicles bearing Interstate Commerce Commission authority numbers, those without ICC authorization but engaged in for-hire travel, and those engaged in private hauling, in the summer of 1955

Vehicle type	ICC authority vehicles	Other for-hire vehicles	Private haulers	All vehicles
	Miles	*Miles*	*Miles*	*Miles*
Single-unit trucks	27	20	15	15
Truck combinations	120	77	75	91
All trucks and combinations	95	30	21	28

Possibly the most interesting information derived from the special study made in 1955 is the data given in table 8 concerning the average trip lengths of freight-carrying vehicles. Trip lengths have been adjusted to eliminate duplications in sampling due to length of trip, since the probability of a trip being recorded, under the method of sampling used, is in direct proportion to the length of the trip. From this table it is seen that the longest trips were made by truck combinations operating with ICC authorization; on the other hand, single-unit trucks in private operation generally made the shortest trips.

Weight Stations Operated

During the summer of 1955, a total of 519 loadometer or pitscale stations were operated in 44 States for the purpose of collecting data concerning vehicle types, weights, and loading practices. During this survey all vehicles (passenger cars, single-unit trucks, truck combinations, and buses) were counted and freight-carrying vehicles were classified according to the number of axles and tire equipment. Approximately 135,000 trucks and truck combinations were weighed, and a record was made of the type of vehicle, the weight of its axles, the spacing (in feet) between each pair of axles, and whether the vehicle was loaded or empty.

The stations used in the 1955 survey were located at the same points as in former years, most of them being at sites operated in the original surveys in the 1936–40 period. From comparable data collected at these locations, trends in travel, loading practices, and carried loads were obtained, which, when applied to former estimates derived from comprehensive surveys, gave current estimates of annual vehicle-miles traveled by loaded vehicles and the carried load. The product of these two factors is the ton-miles of carried load. Data concerning the frequency of overloading and of heavy axle and heavy gross weight occurrence also were made available. The remaining tables and charts in this article have been calculated by means of these trends, or by combining the actual data gathered in the summer survey with vehicle-mileage data developed from trends.

The average weights of loaded and empty trucks and truck combinations in 1955, according to vehicle types, are given in table 9 for the United States as a whole. From this table it is seen that about 55 percent of all freight-carrying vehicles were loaded, weighing an average of 24,336 pounds; empty vehicles averaged 9,426 pounds, and the average of all vehicles, loaded and empty, was 17,592 pounds. Data for 1955 compared with 1950 show that the weights of loaded single-unit trucks increased over 3 percent in the 5-year period; truck combinations, over 6 percent; and for all loaded vehicles, 5 percent. The weights of all empty vehicles increased 5 percent, although the weights of empty single-unit trucks decreased 2.5 percent and empty truck combinations increased over 9 percent.

The heaviest average weights of loaded single-unit trucks in 1955 were found in the South Atlantic division, with those in the New England division being slightly less. The heaviest average weights of truck combinations were found in the Pacific division, with those in the Mountain division being slightly smaller. The heaviest average weights for loaded trucks and truck combinations of all types were recorded in the Pacific division, with the average in the Mountain division slightly lower. At the other extreme, the lowest average weights for all types of loaded vehicles were in the East South Central division. Average empty weights of vehicles followed an area distribution pattern similar to that for the average loaded weights.

Relation of Payload to Total Weight

If it is assumed that the average empty weight of loaded vehicles of a given type is the same as the average weight of empty vehicles of the same type, then subtracting average empty weight from average loaded weight gives average carried load for the vehicle type. On this basis, the average loads carried in vehicles of different types and the relation of these loads to the average loaded weights of the vehicles are as shown in table 10. Thus, in general, the heavier the vehicle type the larger the proportion of the gross weight that consists of carried load or payload. The payload for trucks and combinations with three or more axles averages about one-half of the total weight of the loaded vehicle, whereas for two-axle vehicles it averages much less.

Travel by Loaded and Empty Trucks

The relation of travel by loaded and empty trucks and truck combinations has changed considerably since the prewar period. From 1936 to 1941, between 60 and 65 percent of the single-unit trucks were loaded. When war was declared and driving restrictions were invoked, many small truck owners found it advantageous to use their vehicles for general transportation purposes. Thus, it soon developed that the lighter weight trucks were being used more frequently for personal transportation than had previously been the case.

During the war period, the larger as well as the smaller single-unit trucks were used for the transportation of goods on less than half of their travel. Following World War II, the popularity of the lightweight trucks as a means of personal transportation continued, and in 1950 only about 47 percent of their travel involved carrying a load. The corresponding figure for 1955 was 48 percent. At no time since the war has travel by loaded single-unit trucks equaled that of empty vehicles of this type.

For truck combinations, the relation of

Table 9.—Average weight of loaded and empty trucks and truck combinations, by vehicle types, in the summer of 1955

Vehicle type	Weight loaded	Percentage loaded	Weight empty	Percentage empty	Weight loaded and empty
Single-unit trucks:	*Lbs.*		*Lbs.*		*Lbs.*
Panel and pickup	5, 421	37. 47	4, 173	62. 53	4, 641
Other 2-axle, 4-tire	6, 317	55. 16	4, 865	44. 84	5, 666
Other 2-axle, 6-tire	14, 672	61. 16	8, 572	38. 84	12, 302
3-axle	30, 471	60. 05	14, 765	39. 95	24, 196
Average	11, 241	48. 38	5, 755	51. 62	8, 409
Truck combinations:					
Truck-tractor and semitrailer	42, 695	68. 24	21, 482	31. 76	35, 959
Truck and trailer	62, 428	67. 82	27, 232	32. 18	51, 101
Average	44, 155	68. 21	21, 912	31. 79	37, 085
Average, all trucks and combinations	24, 336	54. 77	9, 426	45. 23	17, 592

Table 10.—Average carried loads by trucks and truck combinations in the summer of 1955, in relation to the average loaded weights of such vehicles

Vehicle type	Average loaded weight	Average empty weight	Average carried load	Relation of carried load to loaded weight
Single-unit trucks:	*Lbs.*	*Lbs.*	*Lbs.*	*Pct.*
Panel and pickup	5, 421	4, 173	1, 248	23.0
Other 2-axle, 4-tire	6, 317	4, 865	1, 452	23.0
Other 2-axle, 6-tire	14, 672	8, 572	6, 100	41.6
3-axle	30, 471	14, 765	15, 706	51.5
Truck combinations:				
Truck-tractor and semitrailer	42, 695	21, 482	21, 213	49.7
Truck and trailer	62, 428	27, 232	35, 196	56.4

travel between loaded and empty vehicles has been fairly uniform throughout the entire period. In 1936, 72 percent of the travel by these vehicles involved the carrying of goods, as compared with 69 percent in 1950, and 68 percent in 1955.

Carried Loads Increase

Average loads carried by trucks and by truck combinations have increased slowly each year. The largest increase for single-unit trucks, 1.86 to 2.29 tons, took place between 1936 and 1941. Since that time average carried loads have not changed materially, being 2.31 tons in 1950 and 2.47 tons in 1955. From 1936 to 1955, the average carried loads hauled by single-unit trucks increased 33 percent. On the other hand, average loads for truck combinations increased rather steadily from 1936 to 1950, and at a reduced rate from that year to 1955. For 1936 the average carried load for combination-type vehicles was 6.90 tons; in 1950, it was 10.62 tons; and in 1955, it was 11.07 tons. The overall increase was 60 percent.

Due to the increased use of truck combinations, the average weight of loads carried by all trucks and truck combinations increased at a still faster rate than for either single-unit trucks or combinations considered separately. Loads hauled by all types of trucks averaged 2.90 tons in 1936, 5.64 tons in 1950, and 5.92 tons in 1955; thus the total increase was 104 percent.

Volume of Freight Hauled

Ton-mileage data provide a measure of the volume of freight hauled on any highway or system of highways. The actual volume of freight carried annually from 1936 to 1955, inclusive, by trucks and truck combinations on main rural roads is shown in figure 5. The chart shows the tremendous growth in ton-miles of freight transported by truck combinations since the beginning of the planning surveys. In 1936 an estimated 13.7 billion ton-miles were transported by combination-type vehicles, and slightly less than 14.3 billion ton-miles were transported by single-unit trucks. By 1940, the combination vehicles were hauling slightly more than the single-unit trucks, and in 1955 the ton-mileage hauled by combinations was three times that of single-unit trucks.

The growth in ton-mileage by single-unit trucks and truck combinations from 1936 to 1955 is illustrated in another manner in figure 6. Ton-mileage is the product of the vehicle-mileage traveled by loaded vehicles and the average tonnage carried by each vehicle. This chart shows the changes that have taken place in each of these factors. The horizontal scale measures the vehicle-mileage for loaded vehicles of each type, and the vertical scale measures the average carried load. Ton-mileage, the product of these two factors, is represented by the areas of the rectangles.

For single-unit trucks, the increase in ton-mileage from 14.3 billion in 1936 to 38.5 billion in 1955 came about mainly through an increase in the mileage traveled by loaded vehicles, since there was very little increase in the average carried load for this class of vehicles, especially from 1950 to 1955. For truck combinations, the increase in ton-mileage from 13.7 billion in 1936 to 115.6 billion in 1955 was the result of a substantial increase in the average carried load and a much greater proportional increase in mileage traveled by loaded vehicles.

It can be seen from figure 6 that almost the entire development of the movement of freight over the highways by the heavy combination-type vehicles has taken place since 1936. In that year, over one-half of the hauling, measured in ton-mileage, was performed by single-unit trucks; in 1955, about three-fourths of it was done by the heavy combination vehicles.

A comparison of the estimated percentage of trucks loaded, average carried load, and ton-miles of freight carried on main rural roads in 1940, 1945, 1950, and 1955 is given separately for single-unit trucks and truck combinations in table 11. The table shows the extent to which the ton-mileage gains were due to

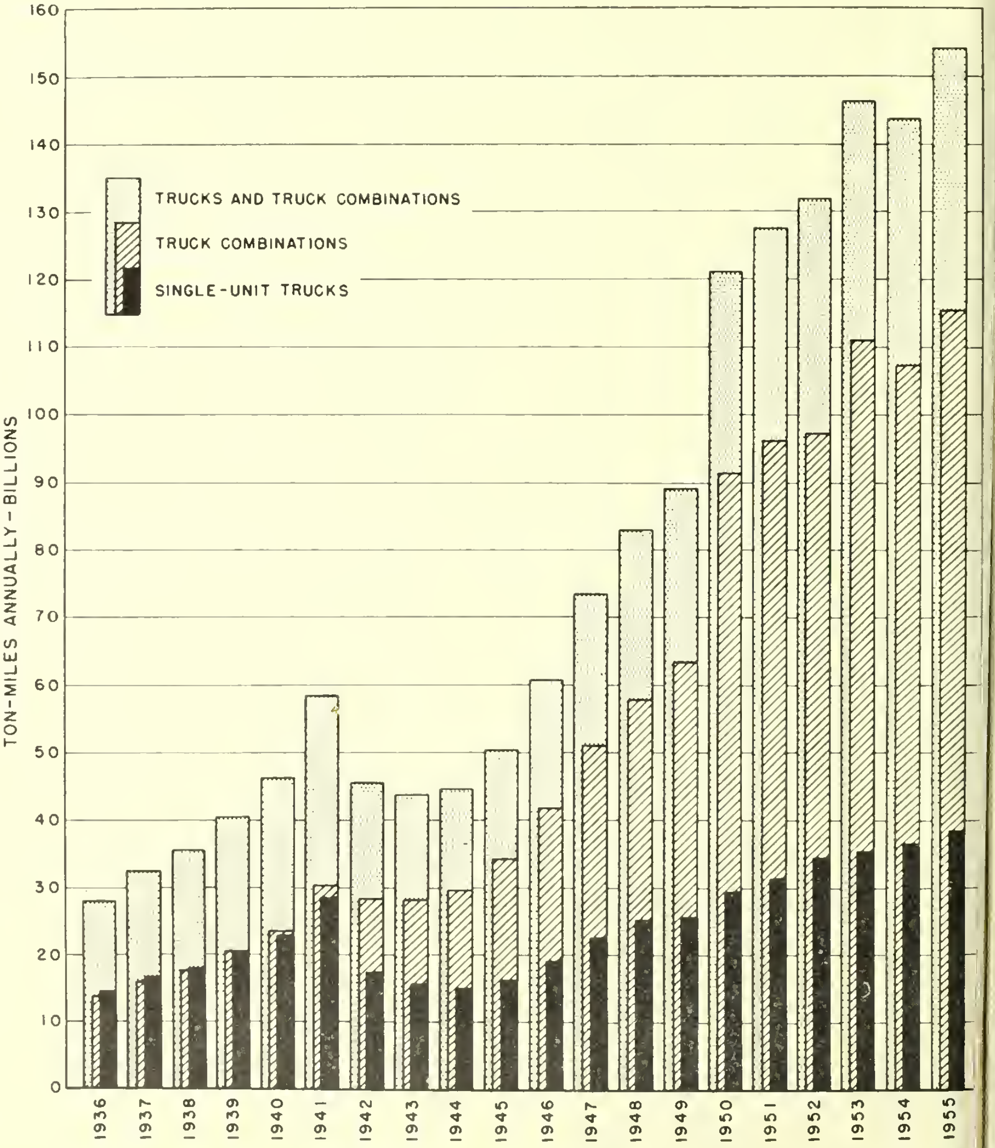

Figure 5.—Ton-miles carried by trucks and truck combinations on main rural road 1936–55.

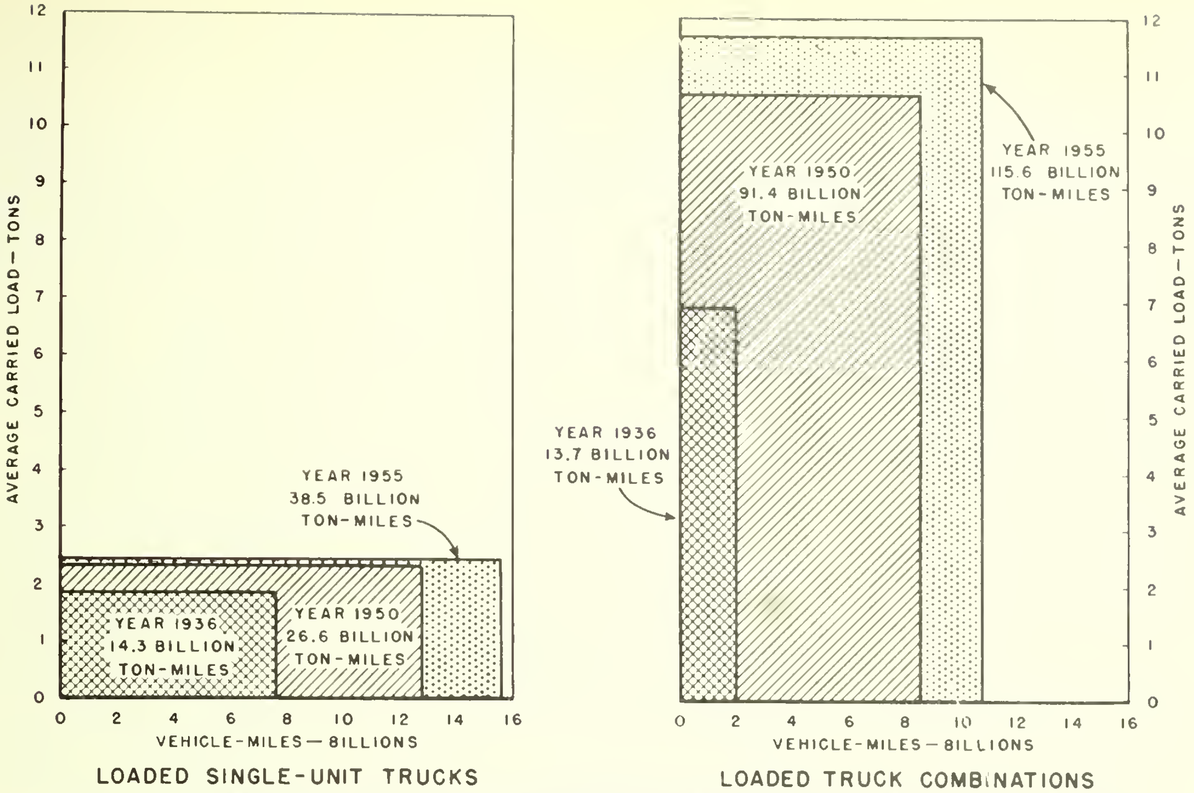

Figure 6.—Vehicle-miles of travel, average carried load, and ton-miles carried by trucks and truck combinations on main rural roads in 1955 compared with 1936 and 1950.

increased loading per vehicle. The increases beyond this point resulted, of course, from increased mileage by loaded vehicles.

Table 11.—Comparison of estimated percentage of trucks loaded, average carried load, and ton-miles carried on main rural roads in 1940, 1945, 1950, and 1955

Year	All trucks and truck combinations			Single-unit trucks			Truck combinations		
	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried
		Tons	*Millions*		*Tons*	*Millions*		*Tons*	*Millions*
1940	65.9	3.32	46,247	64.4	2.13	22,899	71.6	7.41	23,348
1945	55.1	4.84	50,365	49.6	2.40	16,187	69.2	9.31	34,178
1945: 1940 rotio	*.83*	*1.46*	*1.09*	*.77*	*1.13*	*.71*	*.97*	*1.26*	*1.46*
1950	53.9	5.64	121,091	47.2	2.31	29,645	68.5	10.62	91,446
1950: 1945 rotio	*.98*	*1.17*	*2.40*	*.95*	*.96*	*1.83*	*.99*	*1.14*	*2.68*
1950: 1940 rotio	*.82*	*1.70*	*2.62*	*.73*	*1.08*	*1.29*	*.96*	*1.43*	*3.92*
1955	54.8	5.92	154,050	48.4	2.47	38,487	68.2	11.07	115,563
1955: 1950 rotio	*1.02*	*1.05*	*1.27*	*1.03*	*1.07*	*1.30*	*.99*	*1.04*	*1.26*
1955: 1945 ratio	*.99*	*1.22*	*3.06*	*.98*	*1.03*	*2.38*	*.98*	*1.19*	*3.38*
1955: 1940 ratio	*.83*	*1.78*	*3.33*	*.75*	*1.16*	*1.68*	*.95*	*1.49*	*4.95*

Table 12.—Percentage of vehiele-miles of travel, percentage loaded, average carried load, and percentage of total ton-miles carried on main rural roads in 1955 compared with 1950

Vehicle type	Percentage of vehicle-miles of travel		Percentage loaded		Average carried load		Percentage of ton-miles carried	
	1955	1950	1955	1950	1955	1950	1955	1950
Single-unit trucks:					*Tons*	*Tons*		
Panel and pickup	35.37	31.35	37.5	37.4	0.75	0.69	3.05	2.65
Other 2-axle, 4-tire	4.22	2.79	55.2	52.4	1.01	.93	.72	.45
Other 2-axle, 6-tire	25.66	32.63	61.2	55.9	3.53	3.20	17.09	19.06
3-axle	2.51	1.68	60.1	58.3	8.85	7.23	4.12	2.32
All single-unit trucks	67.76	68.45	48.4	47.2	2.47	2.31	24.98	24.48
Truck combinations:								
Truck-tractor and semitrailer	29.86	29.43	68.2	68.9	10.75	10.32	67.53	68.87
Truck and trailer	2.38	2.12	67.8	62.3	15.04	15.32	7.49	6.65
All truck combinations	32.24	31.55	68.2	68.5	11.07	10.62	75.02	75.52
All trucks and combinations	100.00	100.00	54.8	53.9	5.92	5.64	100.00	100.00

In considering the 1955 : 1940 ratios shown in the final entry of the table, for example, it is observed that the ton-mileage hauled by combinations was almost five times as great at the end of this 15-year period as it was at the beginning, while the ratio for the average weight of carried load was 1.49, and the ratio for the percentage of vehicles loaded was 0.95. As shown in table 3, the corresponding ratio for the mileage traveled by all vehicles of the combination type, both loaded and empty, was 3.48, and when multiplied by 0.95 the ratio becomes 3.31 for loaded vehicles. Obviously, most of the enormous increase in ton-mileage was due to increased vehicle-mileage rather than to heavier loading, though the latter factor was of considerable importance in the case of the combination-type vehicle.

For single-unit trucks, the percentage of vehicles loaded decreased more than the average load increased during the period 1940–55. In other words, there was an actual decrease in the average load for vehicles of this type when both loaded and empty vehicles were included in computing the average. The 68-percent increase in ton-mileage carried by single-unit trucks was therefore smaller than the increase in vehicle-mileage, which was 93 percent, as shown by the ratio in table 3.

Comparative information concerning the percentage of vehicle-miles of travel, percentage of vehicles loaded, the average carried load, and percentage of total ton-miles hauled on main rural roads in 1955 and 1950 is given in table 12. Many interesting comparisons can be made from these data. For instance, two-axle, six-tire, single-unit trucks, which are the principal load-carrying single-unit vehicles, account for 26 percent of the total truck travel, yet carry only about 17 percent of the ton-mileage. On the other hand, truck-tractor and semitrailer combinations account for slightly less than 30 percent of the total

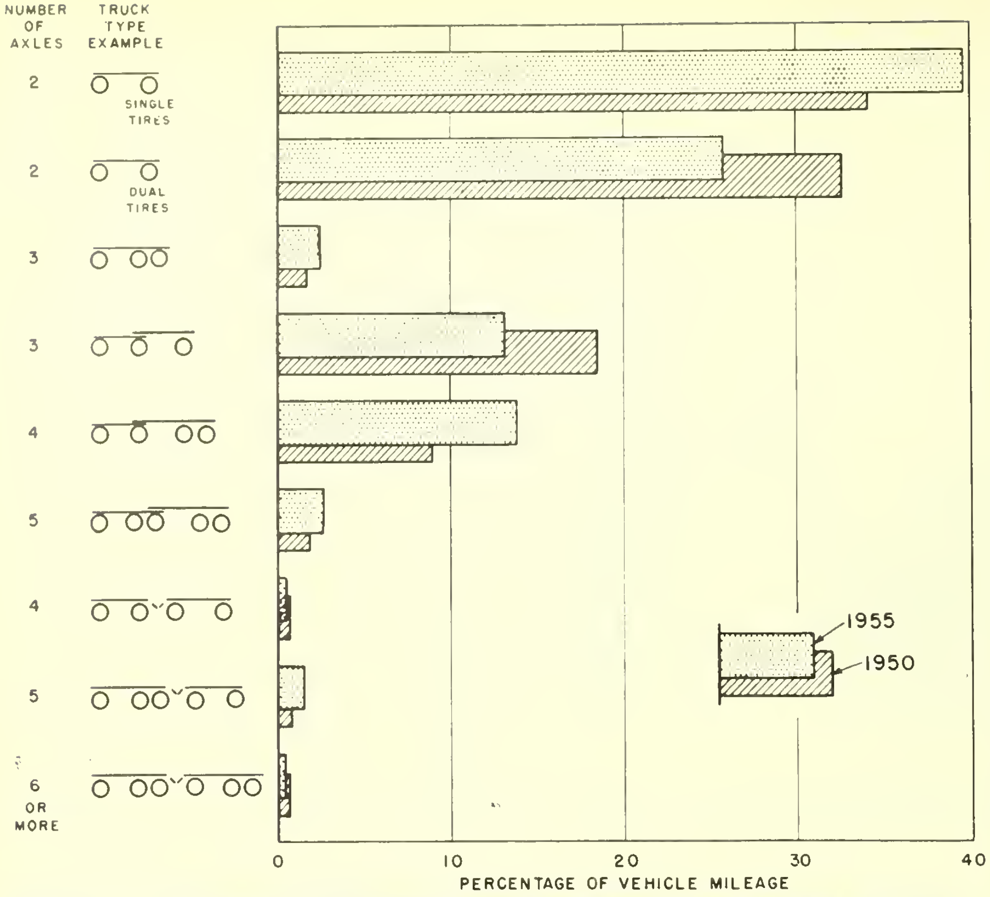

Figure 7.—Percentage of vehicle-miles traveled by various types of trucks and truck combinations on main rural roads in 1955 compared with 1950.

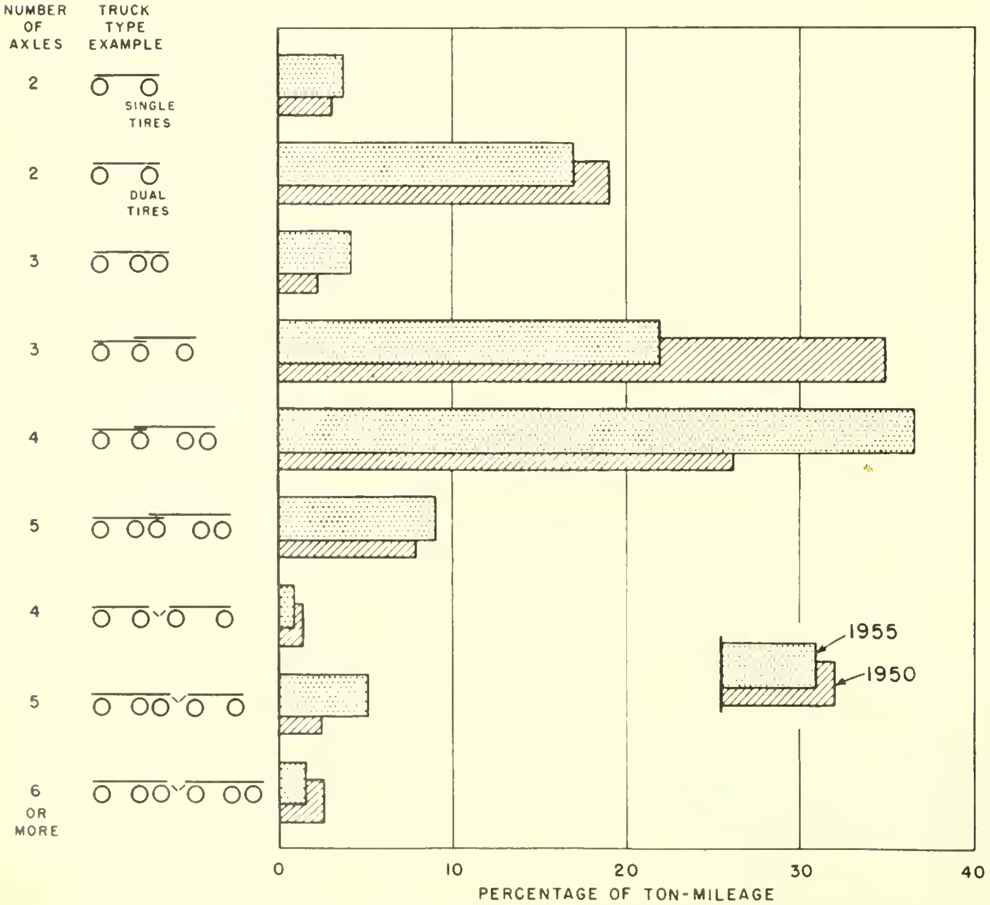

Figure 8.—Percentage of ton-miles hauled by various types of trucks and truck combinations on main rural roads in 1955 compared with 1950.

travel (only a little more than the two-axle, six-tire trucks), but carry almost 68 percent of the ton-mileage or almost four times the amount carried by the smaller vehicles.

The data concerning percentage of vehicles loaded and average carried load are also interesting. Panels and pickups, which carry loads of three-fourths of a ton or less, travel loaded less than 38 percent of the time. Assuming that there is a return trip with no load for each trip with a load, only 75 percent of the travel can be accounted for on the basis of the hauling of goods. Since vehicles are often loaded in both directions, the use of these vehicles as a substitute for passenger cars must be well in excess of 25 percent of their total travel.

The slightly larger two-axle trucks with single rear tires and carrying loads averaging around one ton are loaded about 55 percent of the time and surveys indicate little use for personal transportation. The same is true of the larger single-unit vehicles that operate with a load 60 to 61 percent of the time. Many of the truck combinations are common carriers which seldom are operated empty, inasmuch as they continually pick up and discharge freight. Combinations are found loaded, therefore, in greater proportion than the large single-unit trucks. Many of the latter type carry loads of a one-way variety, such as hauling building materials to construction projects, which tends to reduce the percentage loaded.

Shift to Two-Axle Semitrailers

The percentage of vehicle-mileage traveled by trucks of various types in 1955 compared with 1950 is shown in figure 7. An interesting fact shown by this chart is the great shift that took place in the 5-year period from single-axle semitrailers to those equipped with dual axles. In the earlier period, 18.5 percent of the truck and truck combination travel was performed by two-axle tractors pulling single-axle semitrailers, while travel by the same type of tractor with dual-axle semitrailers amounted to only 9.0 percent.

The 1954 and 1955 survey figures indicated that this relation has changed materially, and the percentage for the dual-axle semitrailers now exceeds the percentage for the single-axle semitrailers. Considerable expansion in the use of three-axle tractors pulling semitrailers equipped with dual axles likewise may be noted, though the percentage for this vehicle type is still relatively low for the country as a whole. These vehicles are found principally in the western divisions, although more are being used elsewhere, especially in the north central divisions. This shift from one-axle to two-axle semitrailers apparently has taken place in order that maximum possible payload can be carried under the weight restrictions in effect in some States.

The percentage of ton-miles hauled by various truck types in 1955 compared with 1950 is shown in figure 8. This chart emphasizes, even more so than figure 7, the shift between 1950 and 1955 from three-axle tractor-semitrailer combinations to four- and five-axle combinations. In 1955 the four-axle combina-

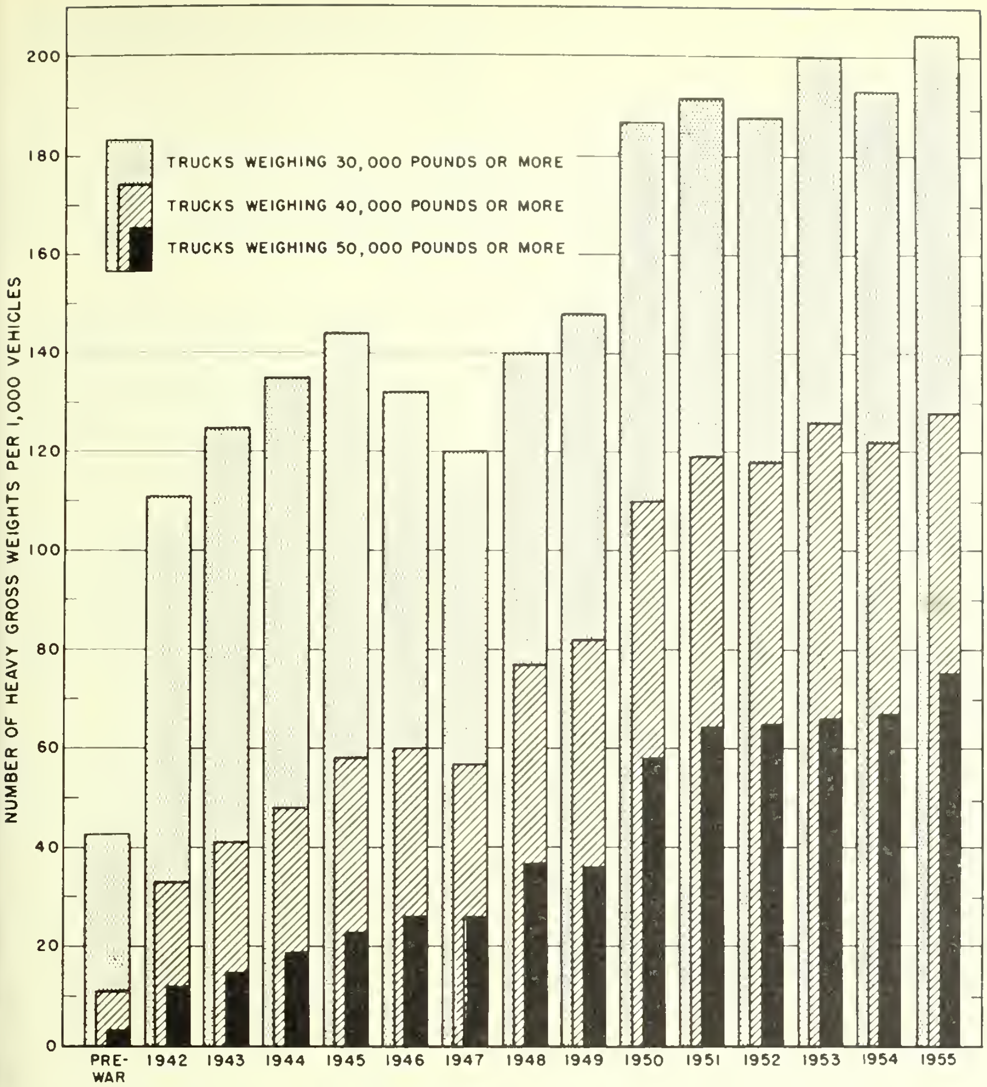

Figure 9.—Number of gross loads of 30,000, 40,000, and 50,000 pounds or more, per 1,000 loaded and empty trucks and truck combinations, on main rural roads in the summers of 1942–55 and a prewar year.

tions accounted for a larger portion of the ton-mileage than any other single vehicle type, whereas in 1950 the three-axle combinations predominated.

Heavy Gross-Load Frequencies

The frequencies of gross loads of 30,000, 40,000, and 50,000 pounds or more per 1,000 trucks and truck combinations on main rural roads in the summers of 1942 to 1955, and in a prewar period (1936–37) are shown in figure 9. During this period of about 19 years the trend of the frequency of vehicles weighing 50,000 pounds or more was consistently upward, although at a generally lower rate of increase from 1951 to 1954, and the frequency of those weighing 40,000 pounds or more was generally upward with temporary drops in 1947, 1952, and 1954. The long-range trend in frequency of weights of 30,000 pounds or more was also upward, but there was a substantial decline from 1945 to 1947 and temporary declines in 1952 and 1954. The 1945 to 1947 decline was quite temporary and was followed by a rapid rise in 1950.

Since 1950 the frequency of weights of 30,000 pounds or more has fluctuated up and down within comparatively narrow limits. While the frequencies of vehicles weighing 30,000 pounds or more and those weighing 40,000 pounds or more were slightly lower in 1954 than in 1953, the 1955 frequencies in each case reached an all-time high. In 1955 the frequency of vehicles weighing 50,000 pounds or more likewise reached a peak. Since the beginning of the planning surveys in the 1936–37 period, the frequency of loads of 30,000 pounds or more increased almost 5 times, those of 40,000 pounds or more increased over 11 times, and those of 50,000 pounds or more increased 25 times.

The 1955 gross-weight frequency data, by vehicle types, are presented in table 13. Since no panels, pickups, or other two-axle, four-tire, single-unit trucks were found in the survey weighing as much as 30,000 pounds, there is no entry for these vehicles in the table. They are included, however, in the total number of vehicles weighed in computing the frequencies for all single-unit trucks and for all trucks and combinations.

In addition to the gross-weight frequencies of vehicles traveling on main rural roads, the variation of these frequencies among the several geographical divisions is interesting. The highest frequency of vehicles weighing 30,000 pounds or more (286 in 1955) was found in the East North Central division, with the Pacific division having the second highest frequency (246). The lowest frequency (158) was in the East South Central division. Similarly, the highest frequency of vehicles weighing 40,000 pounds or more (191) was found in the East North Central division, with the second highest frequency (188) in the Pacific division. On the other hand, the highest frequency of vehicles weighing 50,000 pounds or more (162) was found in the Pacific division, with the second highest frequency (114) in the East North Central division. The lowest frequency for this weight group (22) was found in the East South Central division.

Table 13 shows that the frequency of heavy vehicles is increasing. In the last 5 years the frequency of vehicles weighing 30,000 pounds or more increased 10 percent, vehicles weighing 40,000 pounds or more increased 16 percent, and those weighing 50,000 pounds or more increased 29 percent.

Combining the increases in the frequencies of heavy vehicles with the increases in travel by trucks and truck combinations, the mileage traveled in 1955 compared with 1950 by vehicles weighing 30,000, 40,000, and 50,000 pounds or more increased 31, 39, and 54 percent, respectively.

Heavy Axle-Load Frequencies

The number of axles weighing 18,000, 20,000, and 22,000 pounds or more, per 1,000 trucks and truck combinations, found on main

Table 13.—Frequency of heavy vehicles of 30,000, 40,000, and 50,000 pounds or more, per 1,000 loaded and empty trucks and truck combinations, on main rural roads in the summer of 1955 by main vehicle types

Vehicle type	Number of vehicles, per 1,000 trucks and truck combinations, weighing—		
	30,000 pounds or more	40,000 pounds or more	50,000 pounds or more
Single-unit trucks:			
2-axle, 6 tire	6	(1)	(1)
3-axle	332	102	10
Average	14	4	(1)
Truck combinations:			
Truck-tractor and semitrailer	606	411	231
Truck and trailer	646	518	474
Average	609	419	249
Average, all trucks and combinations	205	128	75
Comparative average, 1954	193	122	67
Comparative average, 1950	187	110	58

[1] Less than 5 per 10,000.

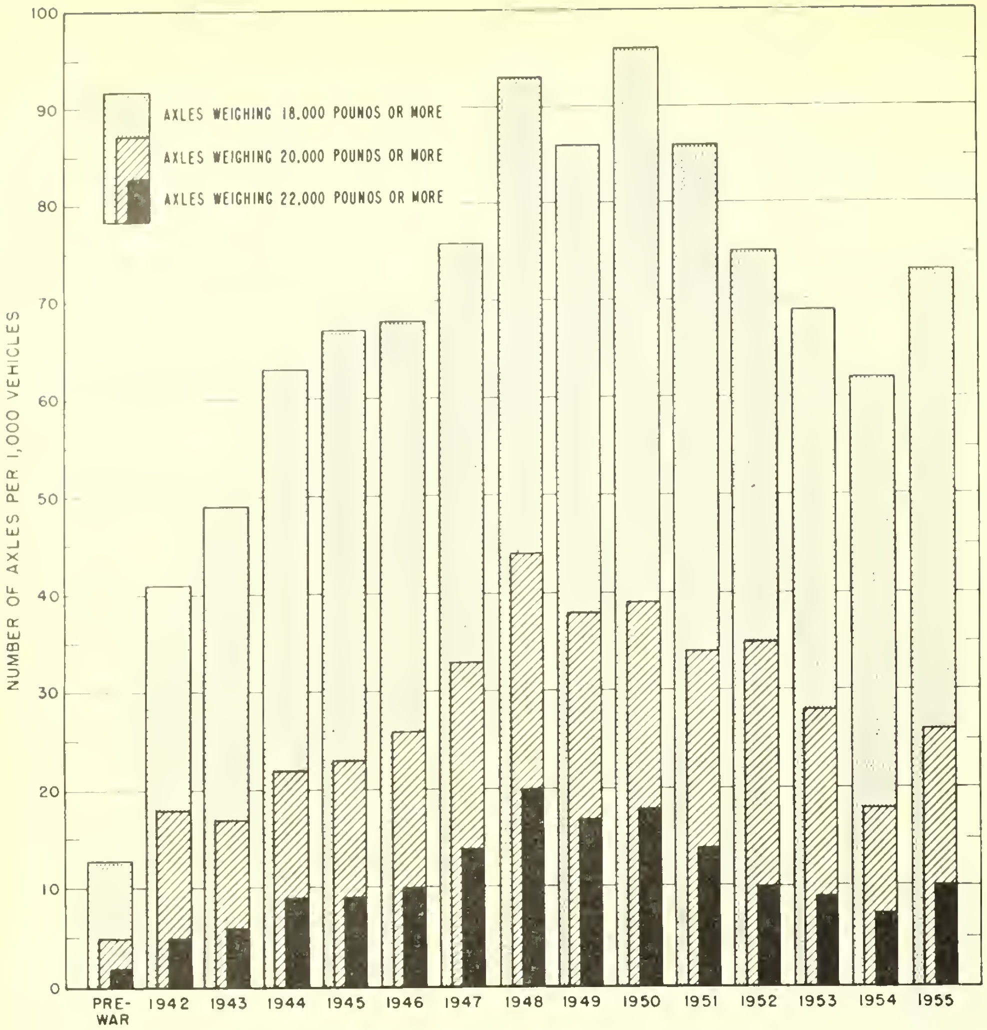

Figure 10.—Number of axles weighing 18,000, 20,000, and 22,000 pounds or more, per 1,000 trucks and truck combinations, on main rural roads in the summers of 1942–55 and a prewar year.

rural roads during the period 1942–55, and a prewar year are shown in figure 10. A most important finding indicated in the chart is the reversal in trend that began in 1951 and continued each year to 1954. The 1955 frequencies show an increase over 1954 for the three weight groups, and in the case of axles weighing 22,000 pounds or more were equal to the 1952 frequency. The number of axles weighing 18,000 pounds in 1950 was more than seven times that in the prewar year, but from the 1950 high there was a drop of 24 percent by 1955. Likewise, the frequency of 20,000-pound axles in 1950 was almost eight times that in the prewar year, but by 1955 the number in this category was two-thirds of that in 1950. The number of axle loads of 22,000 pounds or more increased ninefold from the prewar year to 1950, but dropped 45 percent below that peak in 1955.

Load Distribution Improves

The number of axles weighing 18,000, 20,000, and 22,000 pounds or more for each 1,000 loaded and empty trucks and truck combinations in 1955 are given in table 14 for the main vehicle types. Since none of the two-axle, single-unit trucks with single rear tires was found to have axles weighing as much as 18,000 pounds, that type is not shown in the table. The number of such vehicles counted is included, however, in obtaining the total frequencies for all single-unit trucks and for all trucks and combinations.

In order to give a clearer indication of what is happening on the roads, travel by vehicles with axles weighing 18,000, 20,000, and 22,000 pounds or more was calculated for the period 1950–55. These calculations show that when vehicle-mileage trends are considered, there was an overall downward trend in heavy axle-weight frequencies, notwithstanding the general upward trend in travel and heavy gross-load frequencies that prevailed during the period; decreases in the 18,000-, 20,000-, and 22,000-pound axle-weight categories were 9 percent, 20 percent, and 35 percent, respectively. By a shift to vehicles with a larger number of axles, trucks are hauling more and heavier loads over the highways and yet subjecting them to less frequent applications of heavy and destructive axle loads.

Loads Exceeding Legal Limits

The number of trucks and truck combinations in 1955, per 1,000 loaded and empty vehicles, that exceeded the axle, axle-group, or gross-weight limits in effect in the States or recommended by the AASHO, with comparative figures for the summer of 1950, are given in table 15. The 1955 frequencies, which are generally lower than those for 1950, are also lower than the 1953 figures but somewhat higher than those for 1951, 1952, and 1954. This comparison is similar to the variations in heavy axle-load and heavy gross-load frequencies found in the 5-year period (1950–55) as shown in figures 9–10.

Detailed estimates concerning the overload frequencies in the various geographical areas show that the highest frequency of loads in excess of State legal weight limits in 1955 was in the Pacific division where 71 vehicles in each 1,000 were overloaded to some extent, and 2 vehicles in each 1,000 were overloaded by 20 percent or more. The second highest frequency was found in the West North Central division where 67 vehicles in each 1,000 were overloaded and 3 vehicles in each 1,000 were overloaded by 20 percent or more.

A comparison of frequencies of vehicles loaded in excess of the recommendations of the AASHO shows that the highest frequency of heavy loads was likewise in the Pacific division where 162 vehicles in each 1,000 exceeded the recommended weight limits, and 7 in each 1,000 exceeded these limits by 20 percent or more. The second highest frequency, however, was found in the New England division where 109 vehicles in each 1,000 exceeded the recommendations and 49 in each 1,000 exceeded them by 20 percent or more. The high frequencies of loads exceeding the recommendations in the New England division, are caused by the higher legal axle-load limits in effect in that area. Although

Table 14.—Frequency of axle loads of 18,000, 20,000, and 22,000 pounds or more, per 1,000 loaded and empty trucks and truck combinations, on main rural roads in the summer of 1955 by main vehicle types

Vehicle type	Number of axles, per 1,000 trucks and truck combinations, weighing—		
	18,000 pounds or more	20,000 pounds or more	22,000 pounds or more
Single-unit trucks:			
2-axle, 6 tire	32	12	4
3-axle	83	37	23
Average	15	6	2
Truck combinations:			
Truck-tractor and semitrailer	195	73	29
Truck and trailer	184	21	2
Average	194	69	27
Average, all trucks and combinations	73	26	10
Comparative average, 1954	64	20	8
Comparative average, 1950	96	39	18

able 15.—Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded any of the axle, axle-group, or gross-weight limits in effect in the States or recommended by the AASHO, in the summer of 1955 and the corresponding comparative figures in 1950

Vehicle type	Vehicles exceeding State legal limits						Vehicles exceeding AASHO recommendations					
	Number per 1,000 overloaded	Number per 1,000 overloaded more than—					Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent		5 percent	10 percent	20 percent	30 percent	50 percent
VEHICLES OVER LIMITS IN 1955												
2-axle, 6-tire truck	18	9	5	2	(1)	(1)	27	18	12	5	2	(1)
3-axle truck	64	34	19	11	5	2	98	65	39	15	9	3
Average, single-unit trucks	9	5	3	1	(1)	(1)	14	9	6	2	1	(1)
Truck-tractor and semitrailer	148	83	46	14	5	1	216	152	98	38	16	2
Truck and trailer	209	73	24	7	4	------	442	345	224	59	22	6
Average, truck combinations	153	82	44	13	5	1	233	167	108	39	16	3
Average, all trucks and combinations	55	30	16	5	2	(1)	84	60	39	14	6	1
VEHICLES OVER LIMITS IN 1950												
2-axle, 6-tire truck	20	14	9	4	1	(1)	28	20	15	8	3	1
3-axle truck	70	55	38	13	7	4	77	50	41	18	10	4
Average, single-unit trucks	11	8	5	2	1	(1)	15	11	8	4	2	1
Truck-tractor and semitrailer	187	121	75	30	12	2	249	183	125	56	26	6
Truck and trailer	224	143	92	52	30	4	359	280	191	72	33	10
Average, truck combinations	189	122	76	31	13	2	256	190	129	57	26	6
Average, all trucks and combinations	67	44	27	11	5	1	91	68	46	21	10	3

[1] Less than 5 per 10,000.

he maximum axle-load limit recommended y the AASHO and permitted in most States s 18,000 pounds, the legal limits in States of the New England division range from 20,000 to 22,400 pounds. Consequently, a considerable number of vehicles carrying loads within the State limits may exceed the recommendations of the AASHO by as much as 24 percent.

Experience in Application of Statistical Method to Traffic Counting

BY THE HIGHWAY TRANSPORT RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

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The average daily volume of traffic moving over a particular highway during the year, commonly called the ADT, is a measure of service of that highway to its users. From traffic-counting experience it is known that volumes vary from day to day and month to month, but it is the average daily traffic throughout the year and certain peak volumes to which the highway administrator primarily directs his attention.

To obtain true values of ADT, it would be necessary to count traffic continuously for a year on every section of road between intersections which would be prohibitive from a cost standpoint. On the other hand, a rough estimate of traffic could be made by an experienced traffic engineer, but this method would prove to be too inaccurate. Thus a compromise must be made between accuracy and cost in accordance with need.

The usual practice is to take samples of traffic. Counts of 24- or 48-hour duration are adjusted and expanded into estimates of ADT. The problem is to develop improved procedures for sampling and expanding traffic counts that will produce estimates of acceptable accuracy for design and administrative purposes at minimum cost.

It has been observed in recent years that monthly traffic variations form patterns which tend to persist from year to year on the same road sections. Similarities among patterns permit grouping of rural road sections accordingly, and it has been found that groups thus formed remain substantially intact for several years. This relatively new concept and its application are discussed in this article.

There are about 1,000 continuous-count stations in the United States where traffic-counting machines are permanently installed. The basis for measuring the errors or differences between the estimated ADT and true ADT is provided by the data from these installations. The measure used for evaluating these errors is the standard deviation. That generalizations can be made about traffic volumes on the basis of the standard deviation has been established by the Chi square test of goodness of fit used in this study. The test shows that the errors of ADT estimates follow the normal distribution curve.

Although the application of statistical method in establishing traffic-counting programs, as described in this article, is new, its use in several States has resulted in either a savings in manpower or the obtaining of more accurate and additional information for the same cost.

THE only reason for spending money on counting traffic is to obtain estimates of traffic volumes of acceptable accuracy to be used in establishing structural design and administrative requirements. The efficiency of traffic-counting procedures is then directly related to the degree of accuracy of estimates. Statistical method provides the means of measuring the accuracy of different procedures, thus showing the way to the most efficient one.

Concept of Error of Estimate

One can guess the average daily traffic (subsequently referred to as ADT) at a particular point on a road. But such a guess would not ordinarily be considered acceptable either by those who are responsible for the traffic information or by its users. The reason for the rejection of a guess, of course, is the belief that were the true ADT known it could be considerably different from the guess. If this difference is called an error, then it could be said that a guess may result in too great an error.

It is impractical to obtain true ADT at all points on the road system of a State where such information is needed. It then becomes necessary to arrive at some such value of ADT that in the judgment of the administrator would have error within an acceptable degree of accuracy; thus it is conceded that an estimate of ADT which is based on factual observations, is better than a guess.

Then the task is to find ways of measuring the errors in the estimates of ADT when the estimates are derived by different methods of sampling traffic volumes. In the determination of the ADT, any traffic count for a period of less than one year is considered a sample. The least costly sample from which a sufficiently accurate estimate of ADT can be produced is the most desirable. No single standard exists for the permissible error in estimates of ADT. This decision is usually the responsibility of the administrator.

Sampling of Traffic Volumes

A sample of traffic volume is the number of vehicles passing a particular point on the road during a specific period of time. Samples are obtained by traffic counts. Nowadays machines are used almost exclusively for traffic counts to obtain the total number of vehicles without classification by type.

This discussion deals primarily with traffic counting on rural roads which carry traffic volumes of 500 or more vehicles per day. The cost analysis of a machine traffic count usually reveals that more than 90 percent of such cost is attributed to the wages of the man who installs and removes the machines and the cost of mileage traveled between the places where the machines are located. The places on the road where traffic-counting machines are located will be referred to as stations which may be coverage stations or control stations depending on their purpose.

At a coverage station the traffic is usually counted for 24 or 48 consecutive hours once a year. Here the ADT is estimated by application of factors obtained from control stations. The bulk of traffic-counting stations are of the coverage type. Control stations are of two types: Continuous-count stations where traffic is counted and recorded continuously for a year or several years (about 1,000 stations of this type in the United States), and seasonal control stations where several sample counts are taken in such a manner that the relation between the seasonal or monthly volume of traffic and the ADT can be established. Control stations of both types are used in determining expansion factors for adjusting counts at coverage stations to estimates of ADT.

Traffic counts of 24-hour duration or multiples of 24 hours, such as 48 hours, 72 hours, and so forth, are economical units from which estimates of ADT can be made. Furthermore, such periods of traffic counting are administratively convenient in scheduling the travel of the man who works with the machines. Thus only traffic counts in multiples of 24 hours are considered here. These samples of traffic volumes can be taken once during the year at a particular station or be repeated several times.

Both the duration of traffic counts and the frequency of repetitions have been studied in several States. Only studies of general and fundamental interest are presented in this article. Other studies have been conducted but are not mentioned because they are primarily of local interest.

Use of Statistical Method

Since opinions concerning the accuracy of traffic counts are personal matters and subject to disagreements, it was considered desirable to develop procedures which would permit the measurement of the error in the estimate of ADT and thus appraise it objectively.

The error of estimate of ADT due to a specific method of sampling can be determined only at a few stations where there are permanent machine installations because only there is the true ADT known. Even at permanent installations, the ADT is not absolutely true because of machine failures; the error is comparatively small, however, and is ignored. The ultimate goal is to determine the error at all stations. This cannot be done directly without continuous counts, but the theory of probability in statistical methodology can serve this purpose.

The statistical method, as it has been applied in several States, does not determine the error at a particular station where a traffic volume sample is taken. It does evaluate the size of the error which can be expected at a number of stations without identifying the stations. This means that when traffic counts are made at thousands of stations on the rural roads of a State, it is possible to determine how many stations will have errors of a particular size.

The application of statistical method to a statewide traffic-counting program is relatively new.[1] Only the major sources of errors have been investigated, and means of measuring them have been devised. When these measures were applied to the design of traffic-counting programs, the result was a savings in manpower, or in expenditures, or the obtaining of more accurate information for the same money.

There are, however, yet unexplored sources of error, the evaluation of which can be expected to increase the efficiency of traffic-counting operations even further. It is believed that the discoveries already made do account for the bulk of possible improvements.

Conclusions

All average daily traffic volumes based on samples of traffic counts are estimates. As such they all have errors of estimate. The size of the error depends on the sampling procedure. The cost of obtaining traffic volume information depends on the method of sampling. Statistical method provides the means for measuring these errors in the bulk of rural ADT and vehicle-mileage determinations. It is possible, therefore, to select a procedure for sampling which produces results commensurate with cost.

Experience shows that the estimates of ADT with errors measured by standard deviation of about 10–12 percent for high-volume (over 500 vehicles per day) roads are satisfactory to the users of the data. For this purpose procedures have been developed which are more efficient than the old procedures. The new procedures involve grouping of stations and the predominant majority of road sections on the basis of statistical probabilities of errors of estimates, rather than judgment used in the old procedures. In practical application it means either savings in costs or obtaining more information for the same annual expenditures and yet usually providing more accurate data.

Grouping based on statistical probabilities permits the reduction of both coverage and control operations. Even further improvement in efficiency can be expected in the future when the number and location of continuous-count stations are determined more scientifically for this purpose, and when grouping of road sections according to similarity of seasonal expansion factors will be even better supported by additionally gathered facts.

Fundamental Historical Observations

Persons who deal with traffic counting observed a long time ago that traffic volumes vary in accordance with certain patterns. For instance, at a particular station in a given week, there are about the same number of vehicles counted on each weekday, Monday through Friday, but quite different volumes of traffic appear on Saturday and Sunday. Yet these differences are fairly consistent from one week to another within a month. The traffic volumes will vary from one month to another but the relation of each monthly volume to the yearly volume is about the same over the period of years (data shown in table 9 on page 117 are an example). The recurrence and comparative stability of these elements of pattern gave rise to the idea that if a sample traffic count is adjusted for the variations of pattern, then the sample can be expanded into an estimate of ADT.

The fundamental reasoning which supports the idea of application of statistical method, in the manner here described, to the analysis of traffic-volume patterns and their variations may be enhanced by the understanding of the Gestalt concept and organismic theory which were originally developed in psychological studies.[2]

The idea of expansion of a sample traffic count into an estimate of ADT has been universally accepted and used in this country. There were and still are considerable differences in sampling methods and in the methods of expansion.

As already explained, whether the sample is for a 24- or 48-hour period or even a continuous 7-day count has relatively little effect on cost although the results vary significantly in accuracy. The really big difference in cost of obtaining ADT estimates by several methods lies then in the methods of expansion of counts at individual stations into ADT estimates.

An observation of the greatest importance is that on rural roads where ADT is 500 vehicles or more the pattern of monthly variation of traffic volumes remains pretty much the same for great lengths of consecutive road sections which may or may not be on routes carrying the same route number.

Usual Method of Expanding Samples

The usual procedure for expanding sample traffic counts to ADT has required seasonal control stations which supply the data necessary for computation of expansion factors. The number of seasonal control stations and the schedules of their operations have varied greatly among States. For instance, in one State there are 96 seasonal control stations in rural areas where traffic is counted for 7 consecutive days, 4 times a year, 3 months apart; another State has 560 control stations, each of which is operated for at least 4 consecutive days including a Saturday and a Sunday, 4 times a year, 3 months apart.

In two States the sample counts at coverage stations (operated only once a year and not necessarily every year) are taken simultaneously with control-station counts and each coverage count is then expanded in accordance with the relation of ADT to the traffic volume at the control station on the date of coverage count. The administrator exercises his judgment in relating traffic counts at coverage and control stations. Sometimes slight modifications of this method are used.

In the majority of States composite expansion factors are computed from control-station data. These are grouped on the basis of geographical areas, routes using descriptive correlations such as intercity, recreational, commercial, farm-to-market, and so forth, or routes having similar patterns in the judgment of the administrator. The grouping is usually done without the benefit of statistical evaluation.

The usual procedure is to compute at each seasonal control station the estimated ADT by averaging the traffic volumes during each operation, giving proper weight to weekdays, Saturdays, and Sundays. Then the factors are computed by taking the ratio of ADT to the similar period of coverage-station counts which is generally a weekday. Each such ratio is considered an expansion factor for a particular month. In each group the data from the control stations are used in computing the factors for each month or each season

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[1] *Improving traffic-count procedures by application of statistical method,* by Boris B. Petroff and Robert C. Blensly. Proceedings of the Highway Research Board, vol. 33, 1954, pp. 362–375.

[2] The Gestalt concept can be interpreted as configuration of the whole, the behavior of which can be established by measurement of configuration rather than measurement of the individuals that comprise the whole. In traffic a time pattern (hourly, daily, monthly, etc.) at a point of observation (a station, a road, a State, etc.) may be regarded as configuration of the total (the whole) of the traffic volume. The measurement of these patterns can be obtained from traffic counts at that point but could not be obtained by the study of time-travel patterns of each individual driver who passes that point during the given time period.

Table 1.—Percentage relation of 1954 average daily traffic volumes to average weekday volumes at 25 continuous-count stations in Minnesota

Station number	Group symbol [1]	April	May	June	July	August	September	October	November
152	Ib	102.8	102.1	93.3	90.0	86.3	92.0	102.0	114.3
156	II	97.0	102.2	95.2	99.7	90.0	95.6	102.4	107.1
157	II	103.4	103.3	92.5	95.1	93.0	95.0	101.5	107.0
164	Ia	116.0	105.4	86.7	74.2	74.7	91.8	109.2	120.8
166	Ia	108.8	98.9	87.1	80.1	80.6	92.0	102.0	113.9
170	IV	131.5	106.3	81.4	65.6	68.1	90.6	111.8	129.9
172	Ia	[2] 127.3	105.7	81.9	75.8	81.0	86.7	95.1	107.2
175	Ia	113.3	103.0	85.6	75.9	80.3	89.6	103.6	115.0
179	Ia	105.8	97.5	92.2	91.1	87.1	82.7	99.9	110.4
187	Ia	[2] 124.3	114.4	89.0	72.7	80.5	86.9	114.5	114.6
189	Ib	117.6	103.0	94.2	92.3	93.8	94.4	110.6	119.9
190	Ia	[2] 130.3	105.3	92.0	74.6	85.3	97.9	113.8	118.9
192	Ia	122.5	108.9	101.3	84.9	87.0	96.0	113.3	113.3
193	II	101.5	100.7	96.2	92.8	93.4	97.3	108.5	103.4
194	II	101.4	98.9	87.9	89.1	85.9	95.8	116.1	114.6
195	Ia	112.5	108.8	92.9	84.0	83.4	92.1	107.3	118.6
196	Ia	107.2	101.3	87.2	75.9	76.4	90.5	99.1	114.2
197	Ib	103.9	101.9	91.2	89.6	86.4	92.5	101.2	107.3
198	Ib	105.3	100.5	91.5	86.6	88.3	90.8	99.9	107.3
199	Ia	[2] 132.3	96.1	86.1	85.9	81.1	92.6	103.3	107.1
201	Ia	111.0	103.4	91.6	80.6	86.1	94.2	105.1	112.2
202	IV	131.2	119.6	89.0	64.4	61.3	84.9	109.6	129.5
204	III	188.5	94.9	83.9	73.1	78.6	94.9	140.9	145.1
205	Ib	108.6	105.6	91.4	83.7	81.7	91.9	106.2	108.7
206	II	117.6	106.1	92.5	92.3	85.8	99.6	104.7	109.4

[1] A ±10 percent range of variation among factors in each month was used as a criterion for assigning the group symbol.
[2] Values for various reasons were found unacceptable.

and averaged, thus a mean factor for a month or a season is obtained. This mean factor is then used for the expansion of coverage counts assigned to the group. A number of minor variations of this basic method are also used. Continuous-count recorder data sometimes are included with the seasonal control-station data.

It should be noted in all these methods that judgment rather than objective measurement is exercised in either combining factors or assigning the ratios between coverage and control stations.

New Concept of Objectivity

The important observation that similar patterns of monthly variations of weekday traffic in relation to ADT exist on many road sections already has been mentioned. The practical use of this observation is as follows:

1. Continuous-count station data are obtained from machines located on high-volume highways (ADT, 500 vehicles or more) which are usually on the State designated system. At each such station monthly expansion factors are computed. These factors are the percentages or ratios of the ADT to the average weekday volume of each month.

A ±10-percent range of variation among factors in each month is used as a criterion for the grouping of stations.[3] Since the ultimate objective is the determination of expansion factors for coverage counts, only months during which coverage stations are operated are included in the grouping at continuous-count stations. If during these months the factors between two continuous-count stations do not differ by more than ±10 percent, these two stations are put into the same group.

To expedite this rather tedious process of comparing all the continuous-count stations with each other, the approximation method is used: One station selected by cursory examination as being representative serves as a standard and all other stations are compared with it; all stations having monthly patterns within ±10 percent of the standard are grouped together. Occasionally a station may be put in two different groups. Since in either case it will fall within the ±10-percent range criterion, it makes little difference how it is grouped. Other considerations, such as continuity of route of similar pattern, are then taken into account. For each group for each month compared, a mean factor is computed Thus, if the mean factor were used for expanding the sample instead of the factor for each o the stations in the group, the error introduced due to this substitution would not be greater than ±10 percent. On rare occasions stations having factors beyond the ±10-percent range in only one month have been included in the group.

For example, tables 1 and 2 show the grouping of 25 continuous-count stations in Minnesota. Only the months April through November were used because these are the months when coverage stations were operated Although 17 stations fell into the origina group I, it was conveniently possible to subdivide it into groups I–a and I–b, in each o which the deviations of individual factors from their respective means were even less than th allowed ±10 percent, thus making the mean factors even more accurate.

2. On the State highway map all stations ar indicated with colored pencils in the colo

Table 2.—Monthly expansion factors computed for the 25 continuous-count stations in Minnesota, grouped according to a ±10 percent range of variation of individual factor from their respective group means

Station number	April	May	June	July	August	September	October	November
				Group Ia				
164	1.16	1.05	0.87	0.74	0.75	0.92	1.09	1.21
166	1.09	.99	.87	.80	.81	.92	1.02	1.14
172	(1)	1.06	.82	.76	.81	.87	.95	1.07
175	1.13	1.03	.86	.76	.80	.90	1.04	1.15
179	1.06	.98	.92	.91	.87	.83	1.00	1.10
187	(1)	1.14	.89	.73	.81	.87	1.15	1.15
190	(1)	1.05	.92	.75	.85	.98	1.14	1.19
192	1.23	1.09	1.01	.85	.87	.96	1.13	1.13
195	1.13	1.09	.93	.84	.83	.92	1.07	1.19
196	1.07	1.01	.87	.76	.76	.91	.99	1.14
199	(1)	.96	.86	.86	.81	.93	1.03	1.07
201	1.11	1.03	.92	.81	.86	.94	1.05	1.12
Mean	1.12	1.04	.90	.80	.82	.91	1.06	1.14
				Group Ib				
152	1.03	1.02	0.93	0.90	0.86	0.92	1.02	1.14
189	1.18	1.03	.94	.92	.94	.94	1.11	1.20
197	1.04	1.02	.91	.90	.86	.93	1.01	1.07
198	1.05	1.01	.92	.87	.88	.91	1.00	1.07
205	1.09	1.06	.91	.84	.82	.92	1.06	1.09
Mean	1.08	1.03	.92	.89	.87	.92	1.04	1.11
				Group II				
156	0.97	1.02	0.95	1.00	0.90	0.96	1.02	1.07
157	1.03	1.03	.93	.95	.93	.95	1.02	1.07
193	1.02	1.01	.96	.93	.93	.97	1.09	1.03
194	1.01	.99	.88	.89	.86	.96	1.16	1.15
206	1.18	1.06	.93	.92	.86	1.00	1.05	1.09
Mean	1.04	1.02	.93	.94	.90	.97	1.07	1.08
				Group III				
204	1.89	0.95	0.84	0.73	0.79	0.95	1.41	1.45
				Group IV				
170	1.32	1.06	0.81	0.66	0.68	0.91	1.12	1.30
202	1.31	1.20	.89	.64	.61	.85	1.10	1.30
Mean	1.31	1.13	.85	.65	.65	.88	1.11	1.30

[1] Values for various reasons were found unacceptable.

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[3] "Student's" *t* tests (a measure of significance of the differences of sample means) indicated that this range tends to produce group mean factors that are significantly different.

assigned to the group to which the stations belong.

3. The ratios of ADT to the weekday of the month are also computed for other types of control stations.

4. These control stations are then marked in colored pencil on the same map in accordance with the group to which the control stations appear to belong on the basis of the available expansion factors (usually up to four or six factors per station).

5. When the distribution of continuous-count and seasonal control stations was studied on the map in Florida, New Mexico, Colorado, Idaho, and West Virginia, it was found that the predominant majority in each group fall along continuous road sections which often follow the same route number but not necessarily so. Frequently the consecutive road sections between two continuous-count stations, both belonging to the same group of monthly expansion factors, are along the same route number. However, the route number per se is not necessarily an indication that all of its sections will fall into the same group. Thus, it has been observed that the route of similar traffic-habit pattern is often coincident with a route number but does not have to be. On the other hand, large sections of routes with different numbers very often fall into the same expansion-factor group.

When all stations belonging to the same group are connected by drawing a line of the color chosen for the group, the resultant picture shows the distribution of these highways into groups of monthly patterns, each of which is characterized by the criterion of ±10-percent range of variation of monthly factors at the points of observation. Experience shows that most of the highways in a State distribute themselves by the groups of stations located on them, but usually there is a small mileage of roads that does not seem to fall into any known group either because of the lack of information or for a special reason such as a unique scenic or recreational attraction.

It has been observed that within a few miles of built-up areas of large cities the seasonal factors vary considerably from those of the rural sections of the same road. These factors tend to be closer to the urban factors, which as a rule come closer to unity, than rural for the corresponding months. Since this article deals with the bulk of rural ADT estimates of 500 or more vehicles per day, the rural sections with suburban characteristics are not included in the numerical data presented.

Only continuous-count stations are used in these analyses for the computing of group mean factors since all months are not usually represented at seasonal control stations. The reason why control stations other than continuous-count stations are not used in computing the mean group factors is that they were not properly scheduled originally for that purpose. When distributed by groups, the counts during some of the months predominate, whereas some of the months may not be represented at all. General observations, however, point to the fact that when adequate representation of control counts is available for certain months, then the mean factors for these months are very close to the means based on continuous-count station data alone. Group mean expansion factors could be obtained from properly located seasonal control stations. The continuous-count stations are already in existence, and new installations appear economical in the long run. Furthermore, continuous-count stations serve other purposes for which seasonal control stations are not considered suitable.

Quite frequently a single continuous-count station or the mean pattern of a fewer number of stations than the total number in a group produces practically the same pattern of monthly expansion factors as all stations in the group. This observation gives rise to the possibility of more efficient use of continuous-count stations by moving the excess machines to new locations where the need for information is greater. Seasonal control stations can also be temporarily established for the purpose of proper allocation of a road section to a group. In making these decisions it should be remembered. however, that there are other uses of the continuous-count station data, such as the study of long-range traffic trends. Therefore the machine should be kept at its location unless other considerations show there is no need for it, although it may be superfluous for the purpose of obtaining expansion factors.

The discussion that follows pertains to those roads which can be grouped. These roads represent the great majority. Those few that cannot be grouped require special consideration. Also, as already mentioned, expansion of traffic counts in the suburban developments of larger cities sometimes requires special treatment.

Statistical Significance of Objective Grouping

To make practical use of this objective grouping it is necessary to make the following assumption: That the monthly factors on road sections between two stations falling within the same group, also fall within the same ±10-percent range of variation from the mean factor. This assumption is necessary because the ultimate objective of the grouping is the identification of coverage stations by groups for the purpose of selecting proper expansion factors. In other words, every coverage station should be expanded by the mean-group factor of the group to which its road section has been assigned on the basis of continuous-count and seasonal control stations. This assumption is not arbitrary. The reasoning behind it follows.

For a complete traffic-volume survey of the primary highways in a State the needed number of points in each group at which ADT estimates should be obtained can be assumed to be equal to the number of road sections between intersections—one point for each road section. Thus in each group a total of several thousand stations, either coverage or control, can be expected. This possible total number of stations constitutes the statistical population or universe of stations in each group. In order to obtain such information as the mean monthly expansion factors of a population and the difference between the individual station factors and the mean of the population, it is not necessary to have the data for all the stations in the population. Instead, reasonable approximation can be obtained from a random sample of stations.

From the point of view of statistical theory the sample would be considered to have been selected at random if every one of the thousands of stations in the population had an equal chance of being selected. The continuous-count and the seasonal control stations can be considered a sample taken from the population of stations in the group. However, they were not consciously selected at random. With respect to the concept of random selection these stations can be described as having been selected haphazardly. In locating these stations the primary purpose was to select such places along the highways as were, in the judgment of the administrator, representative of the largest mileage of primary highways. No numerical measures described the meaning of the word representative in the sense of monthly expansion factors.

For this reason there is no awareness of this bias being injected in the selection of stations. Although at times some bias might have been injected accidentally, as observed in the fact that the number of control stations selected does not seem to have any definite relation to the total number of stations under consideration. Thus the sample sizes vary in different pattern groups. It is observed, however, that because the distribution of selected control stations is fairly uniform throughout a State, the sample in each group is approximately proportional to its population. This, however, is not considered too important because each group represents its own statistical population.

In the statistically haphazard distribution of the continuous-count and seasonal control stations there is no knowledge of any bias that could have an effect on the mean values of the monthly factors. Thus these haphazard selections are in effect random. In a random sample of continuous-count stations (of the same number) so selected from the same statistical population, each such sample would produce monthly group mean expansion factors that would not be significantly different from those obtained from the existing selection. The fact that seasonal control stations fall into the same pattern of monthly factors and are along the same habit-pattern routes, as established by grouping of continuous-count stations, tends to demonstrate the validity of the group mean factors.

Understanding of this discussion is necessary because it introduces the theoretical basis for considering the coverage stations as belonging to the same statistical population as the continuous-count and seasonal control stations. This concept, in turn, then justifies the application for expansion of the group mean factors computed from continuous-count and/or seasonal control stations to the coverage counts along the route of similar habit pattern (similar monthly expansion factors). This discussion of theory also may

Table 3.—Standard deviation of the differences (errors) between estimated ADT and true ADT on primary rural systems in eight States [1]

State	Number of stations	Period of count	Number of counts in sample N	Standard deviation, in percent S	Number of counts within one S [2]		Number of counts within two S [3]		Indicated equivalent annual savings due to adoption of new procedure
					Theoretical	Actual	Theoretical	Actual	
		Hours							
Georgia	(4)	24	142	10.1	97	95	135	136	Incomplete analysis.
Idaho	9	48	44	7.2	30	28	42	44	$12,000–20,000.
Minnesota	22	48	199	10.8	136	146	189	190	(5)
Montana [6]	12	48	120	9.6	82	85	114	112	(7)
Montana	12	48	123	9.2	-----	(4)	-----	(4)	
Montana	12	48	122	9.9	-----	(4)	---	(4)	
New Mexico	27	48	175	9.5	119	125	166	170	Possible 50-percent savings.
Oregon	17	24	119	12.6	81	78	113	116	(8)
Washington	12	48	84	9.4	57	62	80	78	$20,000–40,000.
West Virginia	(4)	48	106	12.1	72	82	101	100	(5)

[1] Data for West Virginia are for 1953; all other States, 1954.
[2] Theoretical frequency is 68 percent of N.
[3] Theoretical frequency is 95 percent of N.
[4] Data are not readily available.
[5] Predominant number of control-count stations and a few continuous-count stations can be eliminated on roads carrying 500 or more vehicles per day.
[6] Three independent random samples were used in Montana.
[7] Present studies indicate that urban and local rural road counts can now be taken in addition to rural primary highway counts at no extra cost.
[8] Additional rural and urban vehicle-mileage data and average daily traffic volumes on extensions of the urban system can be determined at no extra cost.

be a cogent stimulus for better appreciation of the significance of the statistical measures used.

It has been found that the standard deviations are significant measures of errors of estimates of ADT that can be expected at coverage stations. These errors distribute themselves normally at the conglomerate of locations. It has been demonstrated by Chi square tests that there is no significant bias that results from this assumption.

The foregoing paragraph indicates that with the aid of continuous-count and other control stations the allocation of road sections to groups of similar monthly expansion-factor patterns can be accomplished. Also the experience with seasonal control stations supports the hypothesis that the continuous-count stations are representative of the population so far as monthly expansion factors are concerned. Where it has been properly demonstrated that the expansion of coverage counts can be made on the basis of continuous-count data alone, then the seasonal control stations or any control stations other than continuous-count stations are ordinarily no longer necessary for development of ADT expansion factors. This means a savings in money or effort which can be used elsewhere.

As pointed out before, there can be unevaluated or unusual situations requiring special treatments. Among such special situations are the following: An isolated road leading to a resort area which has no counterpart in the State; a certain area in the State may temporarily attract unusual volumes of traffic during the hunting season and thus appreciably affect the traffic volumes on a relatively small number of roads; the occurrence of such events as football games which may have a brief but pronounced effect on traffic volumes in some localities; and other situations causing instantaneous variations in traffic volumes or patterns. In these relatively few cases special control stations other than continuous-count stations can and should be used.

Standard Deviation

The error that can be expected in the estimates of ADT at the coverage stations is measured from the data at continuous-count stations by taking random samples of counts of 24- or 48-hour duration, or for any period that may be considered by the State. Each sample is expanded into an estimate of ADT by application of the appropriate group mean factor. At the continuous-count stations the true ADT is always known. The difference between the estimated and the true ADT value is termed the error.

Since the continuous-count stations, when grouped, are considered to represent the population of the group, their standard deviation is also the measure of the errors at the coverage stations—the latter being also samples taken out of the same population. The advantage of the standard deviation [4] as a measure of errors is that it gives the size of the error, and also the expectation of how often an error of that size or smaller may occur. The formula of the standard deviation is as follows:

$$S=\pm\sqrt{\frac{\sum E^2}{N-1}}$$

Where:

N=number of random units in sample.
E=percent error of estimate of ADT.

$$E=\frac{X_1-X}{X}\times 100$$

Where:

X_1=estimated ADT.
X=true ADT.

Table 3 shows the results of a study of errors of estimate of traffic volumes in eight States. The last column of the table indicates the extent of annual savings due to the adoption of new procedures. In compiling table 3 all estimates of ADT were based on each unit in the sample expanded by its appropriate group mean factor. Only weekdays were used in the samples.

In some States, standard deviations were computed for counts of different durations. Computations were made either for data expanded to estimates of ADT, or for unexpanded samples which are individual weekday samples compared with the average weekday of the month. Results are shown in table 4.

In table 5 the standard deviations were determined for differences between the individual monthly weekday expansion factors at each continuous-count station and the respective group mean factor.

The effect of repetition of counts at approximately equal intervals during the year was investigated in several States. The results are shown in table 6. The more frequently the counts are repeated during the year, the more accurate are the estimates of ADT, but because each repetition costs as much as the first count, repeated counts usually are considered prohibitive.

A few studies have been made of single samples of 7-day counts and these indicate a

Table 4.—Standard deviation of the differences (errors) between estimated ADT and true ADT on primary rural systems in eight States [1]

State	Period of count	Number of counts in sample N	Standard deviation, in percent S	Number of counts within one S [2]		Number of counts within two S [3]	
				Theoretical	Actual	Theoretical	Actual
Arizona	24 hours	105	8.3	-------	(4)	-------	(4)
Idaho	7 days	44	6.1	30	27	42	42
Montana	7 days	114	8.2	-------	(4)	-------	(4)
Nevada	24 hours	178	11.0	-------	(4)	-------	(4)
Nevada	7 days	191	7.3	-------	(4)	-------	(4)
New Mexico	7 days	172	8.0	-------	(4)	-------	(4)
Oregon	24 hours	119	9.6	81	83	113	112
West Virginia	48 hours	222	8.6	151	154	211	210
Wyoming [5]	7 days	114	12.3	-------	(4)	-------	(4)
Wyoming [6]	7 days	114	8.8	-------	(4)	-------	(4)

[1] In Idaho, Montana, New Mexico, and Wyoming, computations were made from data expanded to estimates of ADT. In Arizona, Nevada, Oregon, and West Virginia, standard deviations were computed from unexpanded samples.
[2] Theoretical frequency is 68 percent of N.
[3] Theoretical frequency is 95 percent of N.
[4] Data are not readily available.
[5] Group mean monthly expansion factors were used.
[6] Group mean weekly expansion factors were used.

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[4] Standard deviation is a measure of dispersion of individual items about their mean value. In estimates of ADT th mean value of errors approaches zero. The numerical valu of error computed by this measure means that approximatel 68 percent of all individual errors included in the computation can be expected to have values as large or smaller tha one standard deviation; approximately 95 percent of all individual errors to have values as large or smaller than tw standard deviations. The percent of all individual error can be obtained for any multiple of the standard deviation

Table 5.—Standard deviation determined for the differences between the individual monthly weekday expansion factors at each continuous-count station on primary rural systems and the respective group mean factor

State	Number of counts in sample N	Standard deviation, in percent S	Number of counts within one S[1]		Number of counts within two S[2]	
			Theoretical	Actual	Theoretical	Actual
Montana	110	6.5	75	72	105	107
New Mexico:						
Group 1	84	5.2	57	62	80	80
Group 2	105	6.6	-------	(3)	-------	(3)
Group 3	48	5.6	-------	(3)	-------	(3)
All groups	237	5.9	-------	(3)	-------	(3)
Oregon	336	7.3	228	256	319	323
West Virginia, 1952	144	6.2	98	110	137	134
West Virginia, 1953	144	9.2	98	108	137	135

[1] Theoretical frequency is 68 percent of N.
[2] Theoretical frequency is 95 percent of N.
[3] Data are not readily available.

able 6.—The effect of repetition of counts on the standard deviation of the differences (errors) between estimated ADT and true ADT on primary rural systems in six States [1]

State	Frequency of count	Period of count	Number of counts in sample N	Standard deviation, in percent S	Number of counts within one S[2]		Number of counts within two S[3]	
					Theoretical	Actual	Theoretical	Actual
Arizona	Quarterly	24 hours	27	4.6	------	(4)	------	(4)
Colorado	do	7 days	63	4.1	43	42	60	60
Florida	do	24 hours	(4)	7.6	------	(4)	------	(4)
Florida	do	48 hours	(4)	6.1	------	(4)	------	(4)
Idaho	do	4 days [5]	21	4.3	------	(4)	------	(4)
Montana	Semiannually	48 hours	109	7.3	73	80	103	104
Washington	Quarterly	7 days	36	4.2	25	25	34	33
Washington	Bimonthly	7 days	48	3.6	33	36	46	46
Washington	Monthly	7 days	24	1.9	16	17	23	22

[1] In Arizona, Florida, and Montana, Saturday, Sunday, and holiday counts were not included; therefore the 24-hour ount means were compared with the true average annual weekday traffic volumes.
[2] Theoretical frequency is 68 percent of N.
[3] Theoretical frequency is 95 percent of N.
[4] Data are not readily available.
[5] Saturday, Sunday, and two weekdays.

ossibility of their practical use when higher ccuracy is desired than that provided by ingle 48-hour counts. The results are shown n table 4 for Idaho, Montana, Nevada, New lexico, and Wyoming. In Wyoming it was ecided to use the weekly rather than the ionthly expansion factors, since the difference n the cost of analysis is negligible. It is ossible that with additional data improveient in grouping could reduce the errors in Vyoming.

An inspection of standard deviations shown n tables 3–6 reveals how similar are the values or similar tests and how small are the standard errors of the means. Both observations ive strength to the soundness of procedure sed in measuring the error of estimates of DT and to the reliability of the results.

Chi Square Test

The Chi square test is used to provide a ieasure of the "goodness of fit" of observed istribution of units in the sample as comared with the theoretical distribution. If he fit is good then for practical purposes it iay be expected that the characteristics of bserved distribution bear similarity to the haracteristics of theoretical distribution, the itter characteristics always being known. he numerical expression for goodness of fit is iven in terms of P (probability). Again from a practical standpoint the fit may be construed to be reasonably good if P is somewhere between 0.05 and 0.95 as obtained from tables of Chi square values.

In the preceding discussion almost every standard deviation shown was accompanied by the theoretical and the observed number of units having errors no greater than one and two standard deviations, respectively. The consistently close agreement between the theoretical and observed values gave impetus to further investigation of the meaning of this agreement. Within recent months a number of Chi square tests were run leading to the conclusion that the errors in estimates of ADT and expansion factors distribute themselves satisfactorily in accordance with the normal curve. The normal curve of distribution of observed values has the familiar bell shape. Its distinctive feature is that the frequency of occurence of either extremely small or extremely large values is relatively small as compared with the bulk of values which are distributed close to the mean value of observations. Some of these observations are shown in table 7.

All the Chi square tests have indicated good fit or a high degree of probability, and if other similar samples had been taken, good fits could also be expected for them. In a practical sense, the importance of this test is that the empirical data follow closely the normal distribution, thus it can be expected that statistical measures which are based on normal distribution are applicable to traffic characteristics which were tested. The conclusion of special importance is that the standard deviations can be used reliably as a tool for the measurement of the size of the error and the frequency of its distribution.

A sample computation of Chi square is given in table 8 and the values obtained are presented graphically in figure 1.

Persistence of Same Group Mean Factors

If, in two or more succeeding years, the same group mean expansion factors from the same stations are obtained and if all such stations fall within the ±10-percent range from the mean, then it appears that the same grouping of road sections can be extended for a number of years. This is probably true of the majority of road sections belonging to a particular group. It is advisable, however, to watch for any local development that may affect some sections.

Recently a few studies were made concerning the perpetuation of the monthly expansion factors and their relation to estimating the ADT.

In Montana, using only 1954 data, the estimated ADT based on weekday 48-hour

Table 7.—Comparison of the results of Chi square tests of the percentage errors in various sampling techniques

State	Year of data	Number of stations	Number of counts in sample N	Standard deviation, in percent S	Values of Chi square x^2	Probability P
Montana [1]	1954	10	119	6.5	5.81	0.30–.50
Montana [2]	1954	10	120	9.6	2.22	.80–.90
Oregon [1]	1954	38	336	7.3	(3)	.10
Washington [4]	1953	12	86	6.9	13.93	.20–.30
Washington [5]	1953	12	88	7.8	9.22	.60–.70
Washington [4]	1954	12	87	8.7	19.32	.10–.20
Washington [5]	1954	12	84	9.4	13.26	.30–.40
West Virginia [1]	1953	16	144	9.2	16.32	.30–.40

[1] Distribution of percentage differences between monthly expansion factors for individual stations and their respective group means.
[2] Distribution of percentage differences between estimates of ADT based on 48-hour counts and true ADT.
[3] Data are not readily available.
[4] Distribution of percentage differences between estimates of ADT based on 48-hour counts and true ADT. Weekday traffic volumes with Friday p. m. counts excluded.
[5] Distribution of percentage differences between estimates of ADT based on 48-hour counts and true ADT. All weekdays included.

Table 8.—Chi square test distribution of errors in 1953 ADT estimates based on 48-hour counts at 12 continuous-count stations in Washington

Class interval	$\frac{X}{S}$	Cumulative frequency observed	f_o	Cumulative frequency theoretical	f_t	f_o-f_t	$(f_o-f_t)^2$	$\frac{(f_o-f_t)^2}{f_t}$
0–1	0.129	11	11	9.2	9.2	1.8	3.2	0.348
1–2	.258	22	11	18.2	9.0	2.0	4.0	.444
2–3	.3877	27	5	26.9	8.7	−3.7	13.69	1.571
3–4	.516	39	12	35.1	8.2	3.8	14.44	1.761
4–5	.645	47	8	42.8	7.7	.3	.09	.012
5–6	.775	55	8	50.0	7.2	.8	.64	.089
6–7	.904	59	4	56.4	6.4	−2.4	5.76	.900
7–8	1.032	63	4	62.0	5.6	−1.6	2.56	.457
8–9	1.161	70	7	67.2	5.2	1.8	3.24	.623
9–10	1.289	72	2	71.5	4.3	−2.3	5.29	1.230
10–11	1.420	75	3	75.1	3.6	−.6	.36	.100
11–12	1.548	80	5	78.2	3.1	1.9	3.61	1.169
12–13	1.678	80	0	80.8	2.6	2.3	5.29	.514
13–14	1.807	82	2	82.6	1.8			
14–15	1.938	83	1	84.2	1.6			
15–16	2.064	84	1	85.5	1.3			
16–17	2.196	86	2	86.7	1.2			
17–18	2.324	87	1	87.2	.5			
18–19	2.456	87	0	87.7	.5			
19–20	2.580	87	0	88.1	.5			
20–21	2.710	88	1	88.4	.3			

$n=13-2=11$; $P<0.70$ $\chi^2=9.218$

counts had a standard deviation of ±9.2 percent. From continuous-count stations the average rate of change in traffic volumes between 1953 and 1954 was determined. In terms of 1954 volumes, the average rate of change was −3.44 percent. This reduction factor was applied to the 1954 sample of 48-hour counts, bringing them to an estimate of the 1953 sample. The group mean monthly expansion factors were computed from the 1953 data. These factors were then applied to the estimated 1953 sample. The estimates of ADT for 1953 were then compared with the true ADT for 1953. The differences (errors of estimate) thus obtained indicated a standard deviation of ±10.5 percent.

This experiment could be interpreted as follows: If in 1953 there was a comprehensive coverage of all primary highways in Montana, but in 1954 all coverage count stations were eliminated and only continuous-count stations were in operation, then an estimate could be made of 1954 traffic volumes at all stations which were operated in 1953. These estimates then could be expected to lose accuracy to the extent of 1.3 percent (10.5−9.2=1.3). All expenditures made for coverage counts could have been eliminated, were it not for special counts that are always needed.

Comparison of monthly group mean factors in Oregon disclosed that approximately two-thirds were either identical for the 2 years or varied by one percent. A few varied by 2 percent. Table 9 shows the comparison of 1954 and 1953 data in Minnesota.

In Washington, 24 weekday samples taken in 1954 were expanded by group mean monthly factors for 1954 and for 1953. The standard deviations of the errors of estimates of 1954 ADT were as follows:

	Standard deviation
Using 1954 factors____	± 7.6 percent
Using 1953 factors____	± 7.5 percent

The comparison shows that if in 1954 there were no control stations from which the expansion factors could be obtained, then the 1953 expansion factors could be applied to the 1954 coverage counts without any practical change in the accuracy of the ADT estimates.

In West Virginia, a comparison of group mean monthly expansion factors for the years 1954 and 1953 showed that they were practically the same.

From these studies the observation is made that the distribution by groups is quite stable. It is also indicated that even if there are or were in these States seasonal control stations in operation, such operations could be discontinued for a period of at least one year. Also, at least one observation measures the effect of discontinuation of coverage counts for one year on the ADT estimates. The discontinuation of coverage counts for periods as long as 5 years has been practiced before. But the effect of such practice has not been measured statistically.

Seasonal Control Stations

In some States there is not enough information for grouping road sections by patterns of monthly group mean factors. For the purpose of such grouping and also for obtaining group mean factors at the end of the first year of operation of the plan, the following procedure has been devised and used:

Highways are grouped in what is expected to be similar patterns of monthly expansion factors. The continuous-count stations provide the basic information. In each group a series of seasonal control stations are estab-

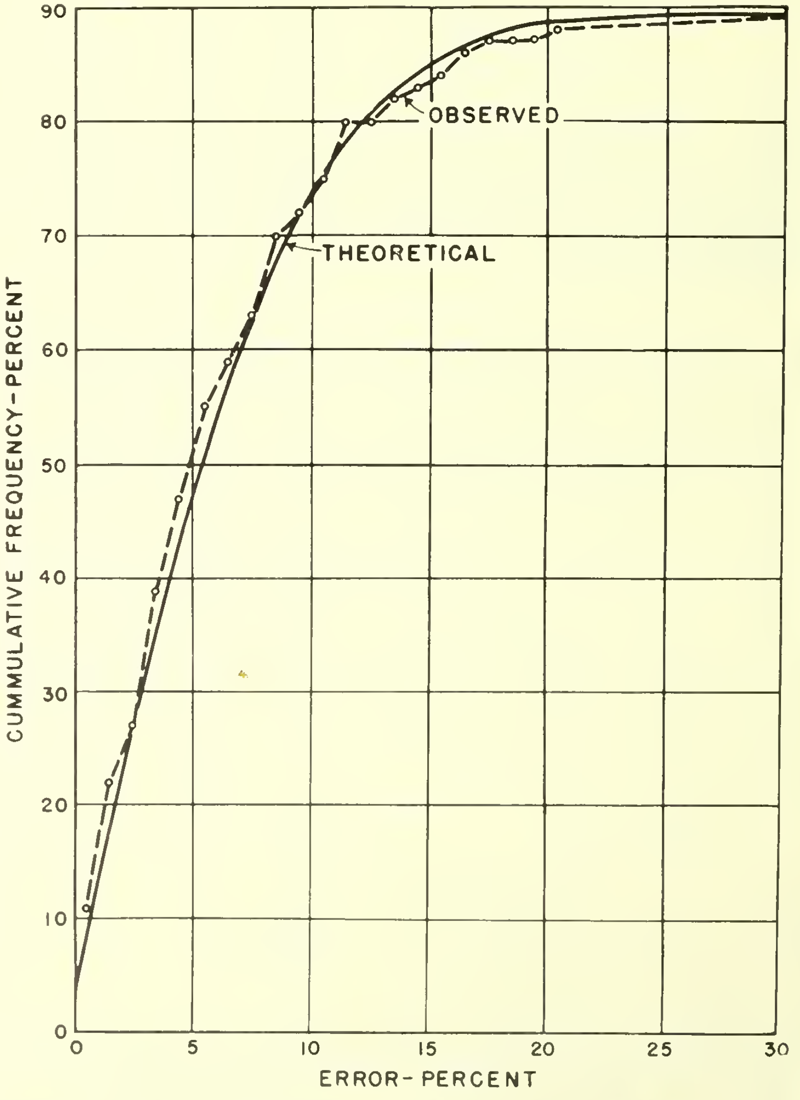


Figure 1.—Comparison of observed and theoretical distribution of errors of ADT estimates, based on 48-hour counts at 12 continuous-count stations in Washington during 1953.

Table 9.—Comparison of the group mean monthly expansion factors for Minnesota and the ranges of variation of individual stations within groups, for the years 1953 and 1954

Number of Stations	Year of traffic count	April		May		June		July		August		September		October		November	
		Mean	Range	Mean	Range	Mean	Range	Mean	Range	Mean	Range	Mean	Range	Mean	Range	Mean	Range
GROUP Ia																	
12	1954	1.12	1.06–1.23	1.04	0.96–1.14	0.90	0.82–1.01	0.80	0.73–.91	0.82	0.75–.87	0.91	0.83–.98	1.06	0.95–1.15	1.14	1.07–1.21
12	1953	1.19	1.06–1.32	1.03	.96–1.11	.88	.81–.95	.80	.74–.90	.83	.74–.89	.91	.84–.97	1.04	.92–1.15	1.14	1.06–1.22
GROUP Ib																	
5	1954	1.08	1.03–1.18	1.03	1.01–1.06	0.92	0.91–.94	0.89	0.84–.92	0.87	0.82–.94	0.92	0.91–.94	1.04	1.00–1.11	1.11	1.07–1.20
5	1953	1.07	.98–1.19	1.00	.94–1.02	.90	.89–.92	.88	.86–.93	.86	.80–.94	.92	.90–.94	1.00	.96–1.06	1.09	1.04–1.15
GROUP II																	
5	1954	1.04	0.97–1.18	1.02	0.99–1.06	0.93	0.88–.96	0.94	0.89–1.00	0.90	0.86–.93	0.97	0.95–1.00	1.07	1.02–1.16	1.08	1.03–1.15
6	1953	1.08	.93–1.22	.99	.90–1.06	.91	.87–.94	.89	.87–.95	.89	.85–.94	.93	.89–.97	1.05	1.01–1.11	1.11	1.06–1.15

lished. It was found that the most practical way is to operate each station for about 1 week, 6 times a year, 2 months apart.

The absolute minimum size sample for obtaining a mean of some statistical significance, in the opinion of many statisticians, is five units in a sample. Five counts per month in each group are then used as a guide. If factors are needed for all 12 months, then the number of such stations in each group is computed thus: 12 months times 5 counts per month divided by 6 counts per station per year equals 10 seasonal control stations per group.

In actually determining the locations of these stations, it is important to select them at random so that the group mean pattern would be representative of the population. In actual experience systematic sampling was used. For this purpose the total mileage of highways in the group was divided by the number of stations which gave the spacing in miles between seasonal control stations. For actual location on the map, the road sections were arranged in a contiguous manner according to section numbers, route numbers, or by adding sections as they appear by moving on the map from east to west or north to south, and so on. The continuous-count stations were included in the control setup. The beginning of the mileage count for locating successive stations was selected at a continuous-count station.

After the first year of operation the correctness of grouping is verified. If need be, for the second year of operations, the seasonal control station setup should be modified by rearranging the grouping and redistributing the seasonal control stations. Ultimately when grouping is completed, the bulk of seasonal control stations can be eliminated.

With some local modification, this general principle was applied in 1951 in Oregon and soon after in Wyoming and Michigan. A considerable number, about 25 percent of the road sections, had to be regrouped after the first year of operation in Oregon. But after 3 years of operation the grouping has been satisfactorily stabilized. The method of seasonal control stations was used at that time, not so much because of the lack of historical information, but because statistical methods and considerations which are discussed in this article had not yet been fully developed.

Table 10.—Standard deviations of the differences (errors) between 1938 estimates of ADT and true ADT for different periods of traffic counts on low-traffic volume roads in Minnesota [1]

Station number	Annual average daily traffic	Σd^2 by stations			
		24-hour count	48-hour count	72-hour count	7-day count
178	118	5,962.5	2,213.6	1,195.1	2,240.3
169	153	1,386.8	904.3	1,287.4	225.4
183	184	436.7	398.4	176.0	190.7
158	220	521.9	1,010.7	861.2	218.9
173	235	2,091.2	507.2	242.7	161.5
151	250	2,618.7	829.0	320.4	388.4
181	275	1,655.5	1,022.7	938.9	952.4
171	278	1,172.2	799.5	460.5	281.4
Σd^2		15,845.5	7,685.4	5,482.2	4,659.0
Standard deviation, S		18.36	14.41	11.45	9.96

[1] ADT estimates based on data for May through October.

Low-Volume Roads

For the same period of traffic counts once the daily traffic volume drops below 500 vehicles, the ADT estimates on such low-volume roads become rapidly less accurate, particularly so on roads carrying less than 300 vehicles per day. The roads with daily volumes below 500 vehicles comprise the great bulk of the mileage of rural roads. To obtain estimates of ADT as accurate as on the high-volume roads, much larger samples would be required at each coverage station. Since there are many more miles of such roads, comprehensive and accurate data would be very costly. In reality, however, it is seldom that data as accurate as on primary roads are necessary for the low-volume roads and, therefore, 24- or 48-hour weekday counts have usually been accepted by the administrators as satisfactory for both high- and low-volume roads.

In table 10 the standard deviations of random samples from their respective monthly means are shown for various time periods. The data are for low-volume roads (less than 300 ADT) in Minnesota for the year 1938. It is observed that these dispersions are much larger than for the higher volume roads.

In designing the procedure for a comprehensive traffic volume survey of low-volume roads, the procedures developed for higher volume roads cannot be used because in some States there are no continuous-count stations or only a very few located on low-volume roads. Thus, grouping by road sections is impracticable.

Since greater errors are permissible on low-volume roads, the recent experience of grouping by geographical areas appears to be satisfactory. In several States the areas were designated on the basis of economic areas established by the U. S. Bureau of the Census. In each area the seasonal control station system is established, except that locations are selected at random in each area rather than systematically on the mileage basis.

An interesting observation was made in Oregon. The mean monthly expansion factors were almost identical in the several areas, although the economic and climatic characteristics were quite different. Also it was observed that although the differences between individual station factors and area mean factors were great, standard deviations ranging from 20 to 25 percent, the group mean factors were quite similar. The result of this observation indicated the possibility of considering the whole State as a single area for computing expansion factors, thus eliminating the cost of operating several seasonal control station systems.

It is felt, however, that existing experience is not sufficiently conclusive to arrive at the decision that a single set of expansion factors for all low-volume roads would be satisfactory in any given State.

A Laboratory Test to Evaluate the Shape and Surface Texture of Fine Aggregate Particles

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported by HARRY M. REX and ROBERT A. PECK
Highway Physical Research Engineers

ONE of the most widely specified characteristics of sand particles comprising the finer fractions of aggregate used in bituminous paving mixtures pertains to the shape of those particles. Thus, specifications usually require that this fine material consist of natural or manufactured sand having sharp, angular grains. It has long been held that materials of such nature are very important to the development of high resistance to deformation in bituminous paving mixtures, and also to the development of a high degree of resistance to skidding in pavement surfaces composed of such mixtures. The development of these properties in the sand fraction is believed to be especially desirable when coarse aggregate materials, having rounded shape and smooth texture, are to be used.

There has been no satisfactory laboratory test for evaluating sands with respect to angularity and texture. Comparison can be made, of course, by means of microscopic examination, but this method is too time consuming for routine analysis, and in addition, the results are obtained by visual observation only.

Measuring Angularity and Texture of Sand

Recently a simple, direct test was developed in the laboratories of the Bureau of Public Roads. It is based upon the principle that smooth textured, rounded sand particles offer less resistance to free flow than do rough textured angular particles.

In this test a specified amount of one-size sand is allowed to flow freely through an orifice and the rate, in terms of seconds per 100 cubic centimeters, is determined. This rate of flow compared with that of Ottawa sand of the same size is taken as a measure of the relative angularity and surface roughness of the sand.

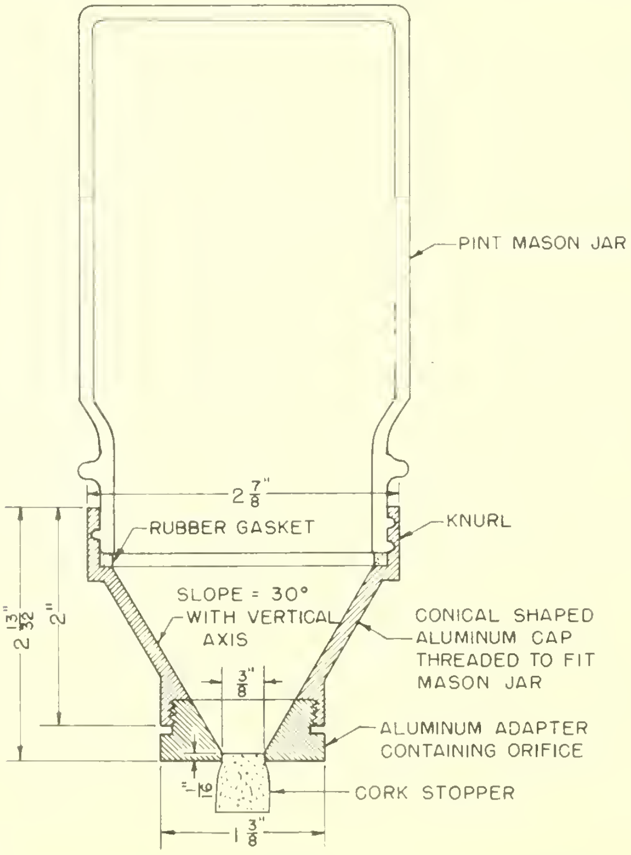


Figure 1.—Cross section of sand angularity test apparatus equipped with ⅜-inch diameter orifice.

The equipment used in making the test consists of a one-pint Mason jar and an aluminum cap in the shape of a frustum of a cone, with a base threaded to fit the jar. An accurately dimensioned orifice is threaded into the center of the top of the cap and is fitted with a cork stopper of suitable size. The apparatus is shown in figures 1 and 2.

Test Procedure

In making the test, 500 grams of dry washed sand, which passes a No. 20 sieve and is retained on a No. 30 sieve, are placed in the glass jar and the aluminum cap is screwed into place. The cork stopper is then inserted, after which the jar is inverted and positioned in a ring stand immediately above a receptacle of a suitable size. The stopper is then removed and the time required for the 500-gram sample of sand to flow through the orifice is measured with a stop watch. Figure 3 shows the test in operation.

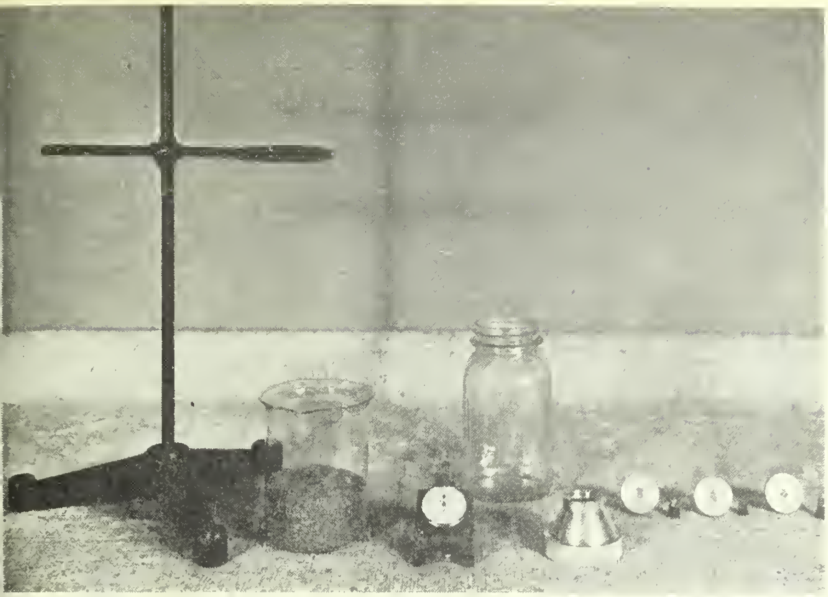
The nine different sands tested are as follows:

1.—Standard Ottawa sand (No. 20–30 sieve size).
2.—Potomac River, Virginia (river sand).
3.—Red Hill granite, Virginia (manufactured sand).
4.—Natchez Trace, Mississippi (bank sand).
5.—College Creek, Virginia (creek sand).
6.—Bar Harbor, Maine (pit sand).
7.—St. Catherine Creek, Mississippi (creek sand).
8.—Canadian River, Oklahoma (river sand).
9.—Black Mountain, North Carolina (creek sand).

The Ottawa sand tested was from the regular laboratory stock of that material. The other sands tested consisted of the No. 20–30 sieve-size fractions of samples that had been sent to the laboratory from time to time. Photomicrographs of small samples of all nine sands, magnified approximately 11 times, are shown in figure 4.

The bulk specific gravity of the sands was determined, using the method described in ASTM Designation C128–42. The results of these determinations are given in table 1.

In the test, the rate of flow is assumed to be a function of the weight of the sample and the


'igure 2.—Sand angularity test apparatus with the three different size orifices shown on the right.

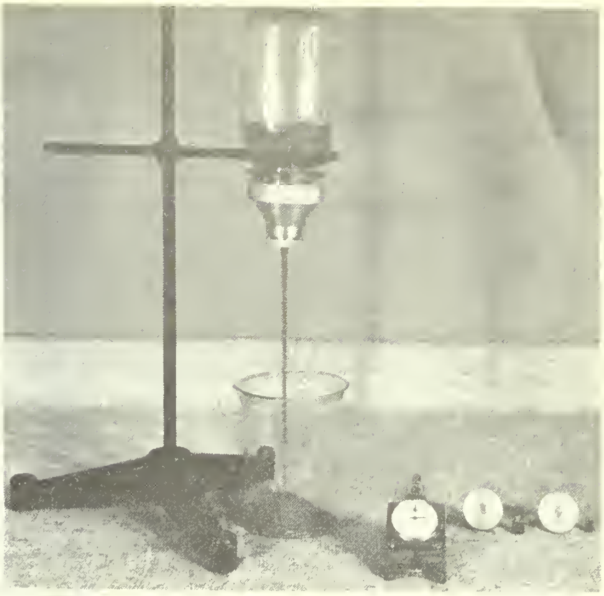


Figure 3.—Sand angularity test in operation.

hape and texture of the particles. If the veight of the test samples is the same in all ases, then the differences in time required for unit volume of sand to flow through an rifice will reflect directly the differences in article shape and texture for various sands.

Evaluating Test Results

Calculations required to convert the test esults into readily usable units for evaluating ands are simple. The number of particles f two or more sands of a single size in a unit f weight is roughly proportional to their pecific gravities. Therefore, in comparing he time required for a unit volume of an nknown sand to flow through the orifice with hat required for the same volume of a standrd sand (in this study the Ottawa sand), a orrection for differences in specific gravity nust be made.

For example, as shown in table 1, it was ound that 25 seconds was required for 500 rams of Ottawa sand to flow through the ⅜-inch orifice. The bulk specific gravity of this sand had previously been found to be 2.66, so there were 188 cubic centimeters of sand solids in the test sample (500÷2.66=188). The rate of flow expressed in terms of seconds per 100 cubic centimeters was, therefore, (25÷188)×100=13.3. The Potomac River sand (No. 2 in table 1) had a bulk specific gravity of 2.60, so there were 192.3 cubic centimeters of sand solids in the 500-gram sample. It was found that it required 33 seconds for the sample to flow through the orifice. The rate of flow of this sand expressed in terms of seconds per 100 cubic centimeters was 17.2. The ratio of these two rates of flow is taken as a measure for comparing the surface characteristics of these two sands, and is expressed as a time index with the standard of comparison taken as unity. Thus, the time index of the Potomac River sand is 1.29 (17.2÷13.3=1.29). The rougher and more angular the sand particles, the larger will be the numerical value for the time index.

The time index values for each of the test sands are given in table 1 and figure 4. These values correlate very well with the discernible surface characteristics of the sand particles.

In the development stage, orifices of ¼-, 5/16-, and ⅜-inch diameters were used. Orifices of 7/16- and ½-inch were considered but not used, because the time of flow for either of these two sizes would be so short that any error in reading the time of flow would appreciably effect the time index. The close agreement between the time index values shown in table 1 indicates that for this particular size of sand the size of the orifice is not critical in respect to ability to differentiate between sands. Since it requires about one-half minute to make the test with a ⅜-inch orifice, any small error in reading the time of flow would not effect the time index, and a longer time would be unnecessary. It is therefore suggested that ⅜-inch orifice be used for the test.

able 1.—Results of tests made with 500-gram sand using orifices of different diameters

Test procedure	Sand designation								
	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6	No. 7	No. 8	No. 9
⅜-INCH DIAMETER ORIFICE									
Total time of flow [1] ... sec	25	33	37	30	32	35	29	28	37
Bulk specific gravity of sand	2.66	2.60	2.76	2.65	2.63	2.58	2.63	2.64	2.72
Solid volume of sand ... cc	188.0	192.3	181.2	188.7	190.1	193.8	190.1	189.4	183.8
Flow rate ... sec/100 cc	13.3	17.2	20.4	15.9	16.8	18.1	15.3	14.9	20.1
Time index	1.00	1.29	1.53	1.19	1.26	1.36	1.15	1.12	1.51
5/16-INCH DIAMETER ORIFICE									
Total time of flow [1] ... sec	43	58	63	50	55	60	48	49	61.7
Flow rate ... sec/100 cc	22.9	30.2	34.8	26.5	28.9	31.0	25.8	25.9	33.6
Time index	1.00	1.32	1.51	1.16	1.26	1.35	1.13	1.13	1.47
¼-INCH DIAMETER ORIFICE									
Total time of flow [1] ... sec	84	115	128	99.5	112	120	96	95.5	126.5
Flow rate ... sec/100 cc	44.7	59.8	70.6	52.7	58.9	61.9	50.5	50.4	68.8
Time index	1.00	1.34	1.58	1.18	1.32	1.38	1.13	1.13	1.54

Recommended Test Procedure

The following test procedure is suggested:

1. Obtain by careful sieving at least 500 grams of the No. 20–30 size sand. Wash and dry this sample.

2. Determine bulk specific gravity of the sample and compute the solid volume for a 500-gram weight of the sample.

3. Place 500 grams of the dry sample in a clean dry Mason jar of pint size. Assemble jar and aluminum cap containing a ⅜-inch orifice fitted with a stopper. Invert assembly and place in a ring stand over a large glass beaker.

4. Begin the determination of flow time by removing the stopper and starting the stop watch simultaneously. Determine the time between the removal of the stopper and the passage of the last sand particles through the orifice. Make a minimum of three determinations and report the average as the time of flow. Individual test results should not vary from the average by more than 2 seconds.

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[1] Average of three determinations.

5. Compute the flow rate in terms of seconds per 100 cubic centimeters of solid volume.

6. Determine the flow rate for standard Ottawa sand which has been carefully sieved to remove all oversized and undersized material. This shall be the standard flow rate with which all other flow rates are to be compared.

7. Compute the time index of the sand under study by dividing its flow rate by the flow rate of Ottawa sand.

Summary

It is believed that this test will be of value in laboratory studies of bituminous mixtures and in evaluating the surface characteristics of sands in the field with respect to specified requirements. The equipment is simple, the operation is practically free from the personal element, and the results obtained by different operators are highly reproducible as indicated in table 2.

Table 2.—Comparison of tests made by two operators using 500-gram sand and orifices of different diameters

Test sequence	Sand designation		
	No. 1	No. 2	No. 3
TIME (IN SECONDS) OF FLOW: 3/8-INCH ORIFICE			
Technician A:			
Test 1	26	33	36
Test 2	25	33	37
Test 3	25	34	37
Technician B:			
Test 1	26	33	37
Test 2	26	33	37
Test 3	26	33	37
TIME (IN SECONDS) OF FLOW: 5/16-INCH ORIFICE			
Technician A:			
Test 1	43	58	63
Test 2	43	58	63
Test 3	43	58	63
Technician B:			
Test 1	43	57	62
Test 2	43	57	62
Test 3	43	57	62
TIME (IN SECONDS) OF FLOW: 1/4-INCH ORIFICE			
Technician A:			
Test 1	83	115	128
Test 2	84	115	128
Test 3	84	115	129
Technician B:			
Test 1	83	114	127
Test 2	83	114	126
Test 3	83	114	127

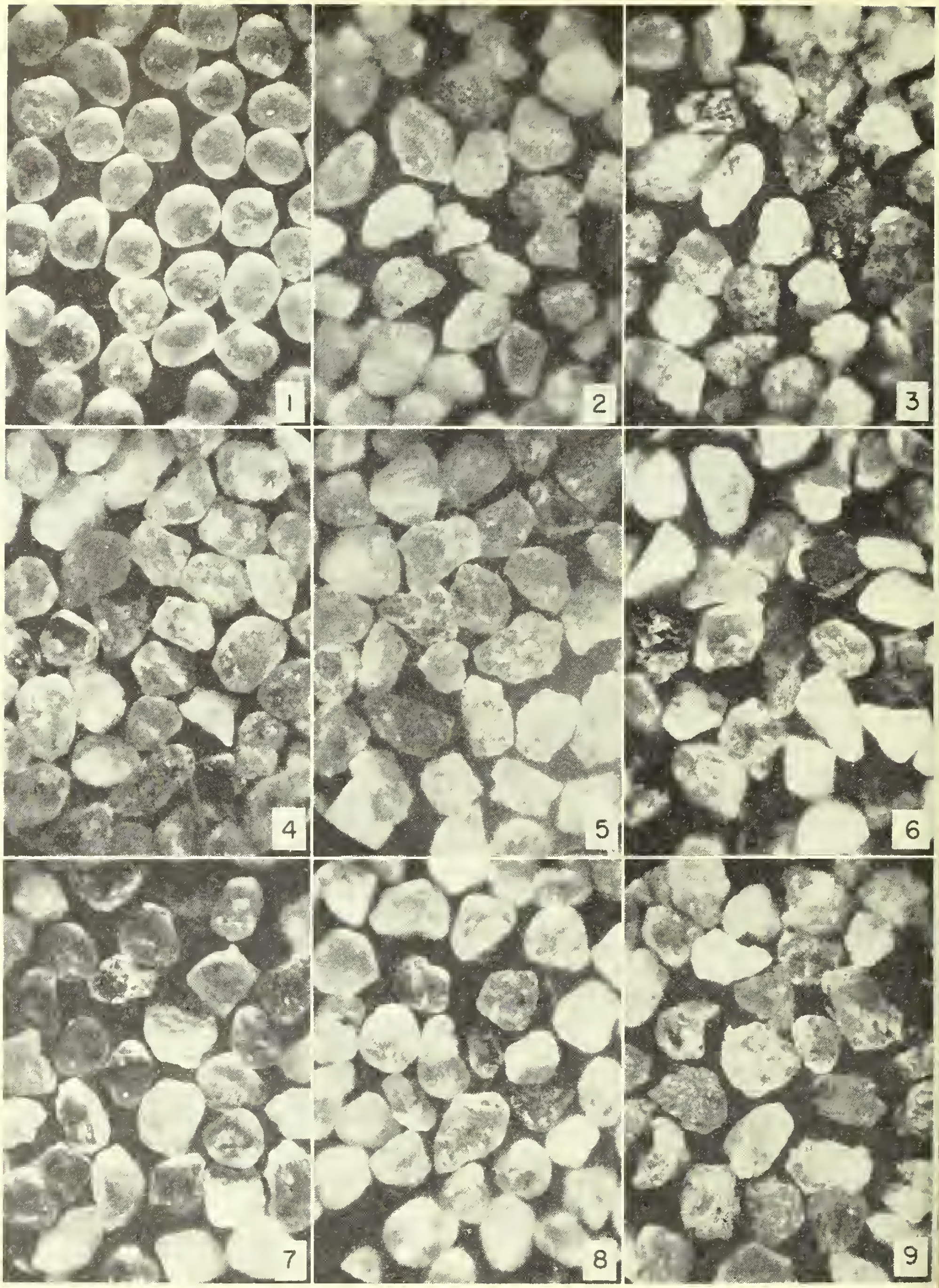

Figure 4.—Photomicrographs of samples of nine sands (X 11.4) having time index values as follows: No. 1, 1.00; No. 2, 1.29; No. 3, 1.53; No. 4, 1.19; No. 5, 1.26; No. 6, 1.36; No. 7, 1.15; No. 8, 1.12; and No. 9, 1.51.

New Publication

The Bureau's *Parking Guide for Cities*, a study of travel habits and parking problems of motorists in urban areas and the characteristics of parking facilities, is now available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 55 cents a copy.

A considerable volume of information has been assembled from these travel-habit and parking studies, and from many other reference sources. Experience and information which have been studied were obtained largely from the medium- and small-size cities, but many of the facts and principles are equally applicable to larger cities.

Collected material on the subject was first published by the Bureau of Public Roads i 1947 in a processed pamphlet, *Factual Guid on Automobile Parking for the Smaller Citie* (now out of print). The text of that pamphle has now been thoroughly revised and expande in the new publication, and an entirely ne section on parking and travel habits has bee added.

U. S. GOVERNMENT PRINTING OFFICE: 1956

A list of the more important articles in PUBLIC ROADS may be obtained upon request addressed to Bureau of Public Roads, Washington 25, D. C.

PUBLICATIONS
# of the Bureau of Public Roads

The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington 25, D. C. Orders should be sent direct to the Superintendent of Documents. Prepayment is required.

ANNUAL REPORTS

Work of the Public Roads Administration:
1941, 15 cents. 1948, 20 cents.
1942, 10 cents. 1949, 25 cents.

Public Roads Administration Annual Reports:
1943; 1944; 1945; 1946; 1947.
(*Free from Bureau of Public Roads*)

Annual Reports of the Bureau of Public Roads:
1950, 25 cents. 1952, 25 cents. 1954 (out of print).
1951, 35 cents. 1953, 25 cents. 1955, 25 cents.

PUBLICATIONS

Bibliography of Highway Planning Reports (1950). 30 cents.
Braking Performance of Motor Vehicles (1954). 55 cents.
Construction of Private Driveways, No. 272MP (1937). 15 cents.
Criteria for Prestressed Concrete Bridges (1954). 15 cents.
Design Capacity Charts for Signalized Street and Highway Intersections (reprint from PUBLIC ROADS, Feb. 1951). 25 cents
Electrical Equipment on Movable Bridges, No. 265T (1931). 40 cents.
Factual Discussion of Motortruck Operation, Regulation, and Taxation (1951). 30 cents.
Federal Legislation and Regulations Relating to Highway Construction (1948). Out of print.
Financing of Highways by Counties and Local Rural Governments: 1931–41, 45 cents; 1942–51, 75 cents.
General Location of the National System of Interstate Highways, Including All Additional Routes at Urban Areas Designated in September 1955. 55 cents.
Highway Bond Calculations (1936). 10 cents.
Highway Bridge Location No. 1486D (1927). 15 cents.
Highway Capacity Manual (1950). $1.00.
Highway Needs of the National Defense, House Document No. 249 (1949). 50 cents.
Highway Practice in the United States of America (1949). 75 cents.
Highway Statistics (annual):
1945 (out of print). 1949, 55 cents. 1953, $1.00.
1946, 50 cents. 1950 (out of print). 1954, 75 cents.
1947, 45 cents. 1951, 60 cents.
1948, 65 cents. 1952, 75 cents.
Highway Statistics, Summary to 1945. 40 cents.
Highways in the United States, *nontechnical* (1954). 20 cents.
Highways of History (1939). 25 cents.
Identification of Rock Types (reprint from PUBLIC ROADS, June 1950). 15 cents.
Interregional Highways, House Document No. 379 (1944). 75 cents.
Legal Aspects of Controlling Highway Access (1945). 15 cents.
ocal Rural Road Problem (1950). 20 cents.
Manual on Uniform Traffic Control Devices for Streets and Highways (1948) (*including 1954 revisions supplement*). $1.00.
Revisions to the Manual on Uniform Traffic Control Devices for Streets and Highways (1954). *Separate*, 15 cents.

PUBLICATIONS (Continued)

Mathematical Theory of Vibration in Suspension Bridges (1950). $1.25.
Model Traffic Ordinance (revised 1953). Out of print.
Needs of the Highway Systems, 1955–84, House Document No. 120 (1955). 15 cents.
Opportunities in the Bureau of Public Roads for Young Engineers (1955). 25 cents.
Parking Guide for Cities (1956). 55 cents.
Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft (1943). $2.00.
Progress and Feasibility of Toll Roads and Their Relation to the Federal-Aid Program, House Document No. 139 (1955). 15 cents.
Public Control of Highway Access and Roadside Development (1947). 35 cents.
Public Land Acquisition for Highway Purposes (1943). 10 cents.
Public Utility Relocation Incident to Highway Improvement, House Document No. 127 (1955). 25 cents.
Results of Physical Tests of Road-Building Aggregate (1953). $1.00.
Roadside Improvement, No. 191MP (1934). 10 cents.
Selected Bibliography on Highway Finance (1951). 60 cents.
Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways, 1956: a reference guide outline. 55 cents.
Specifications for Construction of Roads and Bridges in National Forests and National Parks, FP-41 (1948). $1.50.
Standard Plans for Highway Bridge Superstructures (1953). $1.25.
Taxation of Motor Vehicles in 1932. 35 cents.
Tire Wear and Tire Failures on Various Road Surfaces (1943). 10 cents.
Transition Curves for Highways (1940). $1.75.

MAPS

State Transportation Map series (available for 39 States). Uniform sheets 26 by 36 inches, scale 1 inch equals 4 miles. Shows in colors Federal-aid and State highways with surface types, principal connecting roads, railroads, airports, waterways, National and State forests, parks, and other reservations. Prices and number of sheets for each State vary—see Superintendent of Documents price list 53.
United States System of Numbered Highways. 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

Bibliography on Automobile Parking in the United States (1946).
Bibliography on Highway Lighting (1937).
Bibliography on Highway Safety (1938).
Bibliography on Land Acquisition for Public Roads (1947).
Bibliography on Roadside Control (1949).
Express Highways in the United States: a Bibliography (1945).
Indexes to PUBLIC ROADS, volumes 17–19 and 23.
Title Sheets for PUBLIC ROADS, volumes 24–28.

DEPARTMENT OF COMMERCE - BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF OCTOBER 31, 1956

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$41,831	$42,502	$30,814	515.2	$5,813	$3,726	41.5	$52,423	$30,439	535.4	$100,738	$64,979	1,092.1
Arizona	36,387	7,038	5,788	69.5	5,608	4,108	39.5	9,785	7,255	101.8	22,431	17,151	210.8
Arkansas	47,261	16,540	8,717	454.6	6,372	3,184	86.0	32,317	19,757	445.1	55,229	31,658	985.7
California	145,287	65,195	39,645	274.4	39,872	31,534	41.5	139,769	71,839	203.4	244,836	143,018	519.3
Colorado	50,391	9,265	5,192	107.1	3,111	1,863	19.7	26,203	17,245	152.4	38,579	24,300	279.2
Connecticut	47,488	10,799	8,344	12.9	641	369	.6	9,944	5,154	17.8	21,384	13,867	31.3
Delaware	23,[illegible]91	1,850	1,005	14.8	1,249	645	12.2	8,761	4,327	50.2	11,860	5,977	77.2
Florida	57,915	15,526	7,926	311.5	11,851	6,385	39.9	44,679	23,016	334.3	72,056	37,327	685.7
Georgia	85,940	37,792	20,494	560.0	7,168	3,430	41.2	60,236	29,489	845.6	105,196	53,413	1,446.8
Idaho	36,168	4,796	3,174	46.2	5,901	4,213	31.4	10,964	7,124	160.8	21,661	14,511	238.4
Illinois	131,226	63,234	41,827	506.7	23,139	13,946	68.3	111,304	63,387	681.1	197,677	119,160	1,256.1
Indiana	102,852	22,123	12,020	90.4	7,603	4,101	48.8	39,653	21,758	242.2	69,379	37,879	381.4
Iowa	58,245	28,120	20,222	377.3	11,905	7,766	51.9	32,384	18,710	1,035.3	72,409	46,698	1,464.5
Kansas	53,597	19,760	13,495	702.6	3,908	2,134	59.5	39,421	22,257	1,068.9	63,089	37,886	1,831.0
Kentucky	71,932	6,336	3,259	99.1	3,459	1,749	15.4	40,844	22,047	448.6	50,639	27,055	563.1
Louisiana	54,098	35,231	15,621	141.5	10,427	5,308	10.1	39,308	19,259	305.6	84,966	40,188	457.2
Maine	30,235	7,123	3,572	60.9	1,019	655	5.1	17,550	9,097	112.6	25,692	13,324	178.6
Maryland	20,456	29,504	20,109	99.6	20,203	13,500	19.8	31,537	17,844	130.3	81,244	51,453	249.7
Massachusetts	65,445	32,303	19,808	25.8	13,919	7,666	15.9	51,011	28,382	57.2	97,233	55,856	98.9
Michigan	96,534	56,191	35,418	410.9	15,342	8,802	41.0	75,543	43,542	596.2	147,076	87,762	1,048.1
Minnesota	58,384	22,421	15,505	341.5	7,376	4,837	73.0	54.819	31,116	1,149.3	84,616	51,458	1,563.8
Mississippi	55,191	16,788	8,940	519.6	6,648	4,366	78.3	25,707	13,812	592.0	49,143	27,118	1,189.9
Missouri	81,608	31,451	16,883	1,269.0	6,210	3,801	38.5	80,305	45,084	983.7	117,966	65,768	2,291.2
Montana	57,832	6,698	4,135	141.4	6,148	3,604	74.7	28,184	17,751	425.4	41,030	25,490	641.5
Nebraska	64,006	4,397	2,397	185.7	5,355	3,210	63.7	31,557	16,346	1,000.6	41,309	21,953	1,250.0
Nevada	39,828	3,413	2,849	71.8	1,051	948	.6	11,233	9,511	119.6	15,697	13,308	192.0
New Hampshire	22,019	4,516	2,606	22.6	81	40		11,418	6,770	49.3	16,015	9,416	71.9
New Jersey	92,800	7,206	3,603	55.0	3,901	1,950	1.6	35,892	17,544	50.9	46,999	23,097	107.5
New Mexico	31,165	13,207	11,438	58.2	5,224	3,363	41.7	15,620	10,525	211.6	34,051	25,326	311.5
New York	230,207	35,008	21,367	62.0	47,741	29,659	28.3	274,800	133,030	412.6	357,549	184,056	502.9
North Carolina	92,906	13,401	7,017	171.6	4,865	2,476	33.1	56,294	27,748	706.8	74,560	37,241	911.5
North Dakota	41,993	6,701	3,404	758.7	3,320	1,777	302.5	14,343	7,347	694.0	24,364	12,528	1,755.2
Ohio	102,262	75,258	52,961	196.7	31,640	23,730	44.6	101,537	55,798	143.2	208,435	132,489	384.5
Oklahoma	43,919	46,221	29,213	508.0	6,971	3,567	49.7	50,470	28,319	504.7	103,662	61,099	1,062.4
Oregon	34,139	11,282	8,181	66.1	3,585	3,045	21.8	34,282	22,880	223.3	49,149	34,106	311.2
Pennsylvania	167,286	86,205	52,265	147.6	14,831	7,692	24.7	117,986	59,901	365.3	219,022	119,858	537.6
Rhode Island	20,098	4,497	2,949	5.2	890	717	.1	20,907	11,087	23.9	26,294	14,753	29.2
South Carolina	50,404	17,162	9,383	410.0	3,567	1,965	18.0	24,752	13,363	491.8	45,481	24,711	919.8
South Dakota	42,165	10,327	5,959	358.7	671	376	35.1	17,920	10,330	545.7	28,918	16,665	939.5
Tennessee	81,981	18,123	8,312	299.0	15,366	7,683	60.3	47,107	21,829	450.5	80,596	37,824	809.8
Texas	176,691	19,914	10,914	395.7	45,349	29,978	267.5	121,731	67,051	1,391.5	186,994	107,943	2,054.7
Utah	33,177	6,443	4,697	72.1	664	492	1.9	8,722	6,809	119.9	15,829	11,998	193.9
Vermont	22,684	1,191	596	17.8	381	190	2.1	11,007	6,238	71.1	12,579	7,024	91.0
Virginia	75,538	17,365	9,661	170.1	3,590	2,021	17.7	34,114	17,378	350.2	55,069	29,060	538.0
Washington	56,087	9,260	5,399	105.5	5,867	3,490	63.8	32,007	18,056	211.5	47,134	26,945	380.8
West Virginia	49,164	15,415	8,130	58.3	6,722	3,370	31.6	20,780	10,528	56.7	42,917	22,028	146.6
Wisconsin	83,588	12,675	6,424	118.9	7,751	3,994	50.0	44,578	22,582	339.0	65,004	33,000	507.9
Wyoming	16,013	21,958	18,060	162.3	2,974	1,871	29.8	18,693	13,393	228.4	43,625	33,324	420.5
Hawaii	7,507	1,448	714	3.4	3,701	1,844	6.7	3,298	1,533	2.4	8,447	4,091	12.5
District of Columbia	25,769	11,801	6,931	3.7	111	48		8,458	4,245	1.9	20,370	11,224	5.6
Puerto Rico	11,887	7,825	3,381	26.6	1,148	459	.7	18,973	8,888	56.6	27,946	12,728	83.9
Alaska	15,074												
TOTAL	3,240,411	1,070,805	660,724	11,663.8	452,209	281,647	2,151.4	2,251,130	1,218,720	19,498.2	3,774,144	2,161,091	33,313.4

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FEBRUARY 1957

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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In this issue: Two articles on the use of fly ash in concrete.
(Garden State Parkway in New Jersey)

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Published Bimonthly

Vol. 29, No. 6 *February 1957*

C. M. Billingsley, *Editor*

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Studies Relating to the Testing of Fly Ash for Use in Concrete

BY THE PHYSICAL RESEARCH BRANCH BUREAU OF PUBLIC ROADS

Reported[1] by RUSSELL H. BRINK, Highway Physical Research Engineer, and WOODROW J. HALSTEAD, Chemist

Recent interest in the use of fly ash in concrete for highways has created a need for suitable specifications and methods of test for this material when it is used to replace part of the cement. The effect of fly ash on the strength of portland cement-fly ash mortars is considered to indicate the probable effect on the strength of concrete, and is used in this investigation as a criterion for evaluating other tests of fly ash. The effect of fly ash on the expansion resulting from the alkali-aggregate reaction is also considered. To determine the variations in properties which may be expected and the effect of these variations on the behavior of the fly ash in mortar, 34 fly ashes from 19 different sources were examined.

The results of the tests indicate generally that the strength developed in portland cement-fly ash mortars is related to the carbon content of the fly ash, the fineness of fly ash as measured by the amount passing the No. 325 sieve, and the water required for mortars containing fly ash as compared with the water required for similar mortars without fly ash. No well-defined relation between the individual inorganic constituents of the fly ash and the development of strength of the mortar was obtained.

These tests also indicate that the effect on the strength of mortar of replacing a portion of the cement with fly ash varies according to the cement used.

Tests of mortar bars show that fly ash may be used to inhibit the expansion resulting from the alkali-aggregate reaction. These tests indicate that the amount of fly ash required to reduce expansion below a safe limit varies for different fly ashes.

FLY ASH is a powdery residue resulting from the burning of pulverized coal. In modern power plants this material is collected at the entrance to the stack by electrical or mechanical precipitators to prevent pollution of the air. Fly ash is usually finer than portland cement and consists mostly of small spheres of glassy compounds of complex chemical composition together with miscellaneous materials such as quartz, feldspar, iron oxides, and carbon.[2] Photomicrographs of two fly ashes are shown in figure 1.

During recent years there has been a growing interest by both governmental and industrial organizations in the use of fly ash as an additive to concrete. A number of reports have been published which indicate that beneficial effects may be obtained when fly ash is used to replace a portion of either the sand or the cement (*1, 2, 3, 4*).[3] In an investigation of concrete for the Hungry Horse Dam, Blanks (*1*) reported that fly ash improved the workability and decreased the bleeding of plastic concrete. The segregation of aggregates in concrete was also decreased. In the hardening concrete it was found that a cement-fly ash combination produced less rise in temperature than an equal amount of portland cement. Other beneficial effects mentioned were decreased permeability, better resistance to sulfate attack, and reduced expansion resulting from the reaction between the alkalies of the cement and certain types of aggregate.

Fly ash has been used most extensively in mass concrete, particularly large dams, where many of the changes it produces in concrete are of value. As fly ash is less expensive than cement, its use to replace a portion of the cement has resulted in significant savings in the cost of certain projects. The possibility of reducing the cost of concrete for highway construction is responsible for much of the current interest by highway agencies in the use of fly ash. Interest has also been shown in its use at locations where the alkali-aggregate reaction is a problem.

Regardless of the primary reason for using fly ash in concrete, the effect of the admixture on the strength of the concrete must be considered. When fly ash is used to replace a portion of the cement, the strength of the resulting concrete is usually reduced at early ages but with adequate curing it may eventually equal or exceed that of concrete without fly ash (*1, 2*). Fly ash contributes to the strength of concrete at later ages because it is

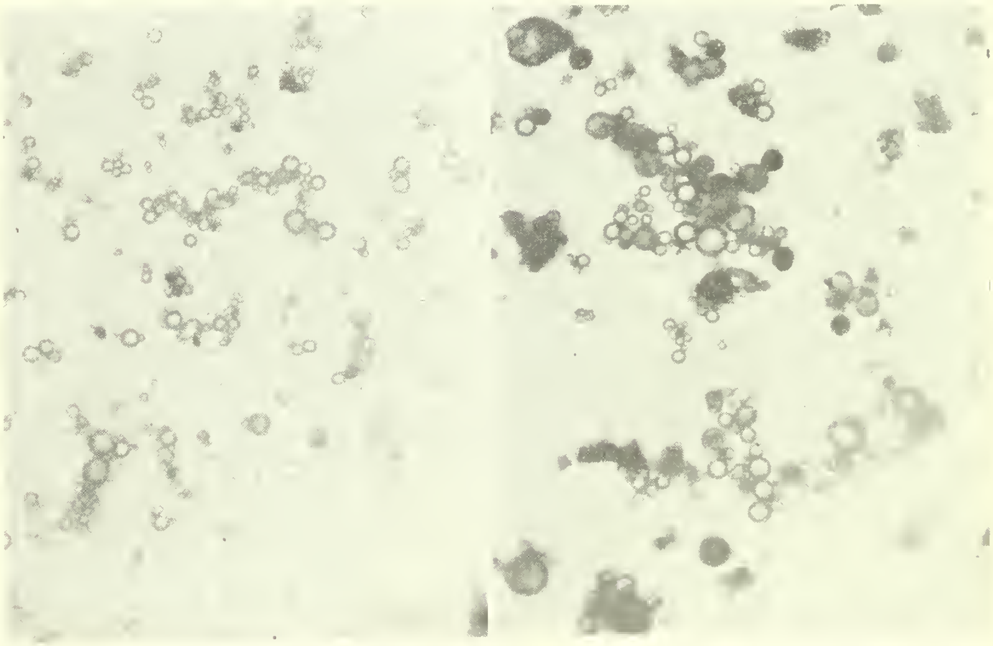

Figure 1.—Photomicrographs (×390) of two samples of fly ash having the following carbon content: No. 1 (left), *0.2 percent; and No. 15* (right), *5.5 percent.*

---

[1] This article was presented at the 59th Annual Meeting of the American Society for Testing Materials, Atlantic City, N. J., June 17–22, 1956.

[2] A discussion of the nature and source of these materials and the chemical changes that occur during the burning of the coal is included as Appendix B on p. 140.

[3] Italic numbers in parentheses refer to the list of references on p. 141.

a pozzolanic material. A pozzolan is defined in ASTM Standard Definitions of Terms Relating to Hydraulic Cement, C 219–55, as "a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties." The reaction between fly ash and the lime released by the hydration of cement develops slowly, and a minimum of 30 days may be required before the strength developed in concrete by this reaction will compensate for the initial loss in strength resulting from the lower cement content.

Fly ashes from different sources vary considerably in both chemical composition and physical properties due to differences between the coals from which they are derived and the conditions of burning. Consequently, it is to be expected that their behavior in concrete will not be uniform. This investigation was conducted primarily to study the pozzolanic behavior of fly ash as shown by its effect on the strength of cement mortar. The strengths developed in mortars containing fly ash were used to study the relations between pozzolanic activity and the various chemical and physical properties of fly ash. Comparisons were also made between the effect of fly ash on the strength of mortar and the results of other tests involving reaction of fly ash with lime or sodium hydroxide.

At the time this investigation was undertaken, it was generally recognized that finely divided, low-carbon fly ashes were useful as admixtures to concrete. However, there was no general agreement as to the limits which should be placed on these properties or the extent to which these properties alone could be relied upon for selecting fly ashes. It was hoped that the information obtained in this investigation would provide a suitable basis for the selection of fly ashes for use in concrete. A study was also made of the effectiveness of fly ash in reducing expansion resulting from the alkali-aggregate reaction. These results were considered separately from the general behavior of fly ash as a pozzolanic material.

To have fly ashes with a wide range of physical and chemical properties represented in this investigation, 34 samples produced by 12 different concerns at 19 individual plants were obtained. Fly ashes from the sources represented by samples 1, 3, 5, and 16 had been used in concrete with good results, but the use or quality of the other samples was not known. Four of the samples (1, 3, 14, and 29) were also used in an investigation in which the effect of fly ash was determined directly on concrete specimens. The results of that study are reported in a companion article.[4]

Conclusions

The results of this investigation show that many factors affect the development of strength in a cement-fly ash mortar as a result of pozzolanic action. It was found that this action is influenced not only by the character of the fly ash but also by the properties of the cement. A general relation was found between pozzolanic activity and several other properties of fly ash.

The following general conclusions appear to be justified:

1. The effect on mortar strength of replacing part of the cement with fly ash varies with the cement used.

2. Compressive strength tests conducted on cement-fly ash mortars at various ages and with different cement contents indicate that ultimately fly ash has the greatest proportional benefit to lean mixes, even though tests at 28 days generally show the reverse to be true.

3. The carbon in fly ash lowers the strength of mortar because it increases the amount of water required to obtain a workable consistency, and reduces the amount of the pozzolanically active constituents of the fly ash. However, only a general relation between carbon content and mortar strength was found since other properties of fly ash also affect the development of strength of mortar due to pozzolanic action.

4. Specific surface determined by the air permeability method is often used as a measure of fineness in fly ash specifications. No relation was found between the strength of cement-fly ash mortars and fineness determined by this method.

5. In general, the strength of cement-fly ash mortars varied with the fineness of the fly ash as determined with the No. 325 sieve, the finer fly ashes being associated with the higher strengths.

6. A general agreement was found between the effect of fly ash on the strength of mortar and the amount of water required to prepare the mortar. If the addition of fly ash required an increase in the amount of water needed to prepare mortar of a stated consistency, a decrease in the strength of the mortar resulted.

7. No agreement was found between the amount of any single inorganic constituent in fly ash and the efficacy of the fly ash as a pozzolanic material.

8. A method involving reaction of silica with sodium hydroxide has been used with some success for determining the quality of pozzolanic materials. This test was not found suitable for testing fly ash because the lime or calcium sulfate present in fly ash prevented the normal course of the reaction.

9. A lime absorption test, involving reaction of fly ash in a lime solution, gave no reliable data for predicting the pozzolanic activity of fly ash.

10. A test for the compressive strength of lime-fly ash mortars cured at a temperature of 130° F. failed to give results agreeing satisfactorily with the strengths of cement-fly ash mortars. It is possible that changes in the details of the lime-fly ash mortar test may improve the reliability of the test, and a further study of this method is considered advisable.

11. Fly ash will inhibit the volume change caused by the alkali-aggregate reaction if

Table 1.—Chemical analysis of fly ash samples (oven-dry basis)

Identification number	Loss on ignition, 600° C.	Carbon	Silicon dioxide SiO_2	Ferric oxide Fe_2O_3	Aluminum oxide Al_2O_3	Calcium oxide CaO	Magnesium oxide MgO	Sodium oxide Na_2O	Potassium oxide K_2O	Total alkalies as Na_2O	Sulfur trioxide SO_3	Carbon dioxide CO_2
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
1	1.2	0.2	47.1	19.2	18.2	7.0	1.1	1.80	2.15	3.21	2.8	0.01
2	1.0	.5	48.4	19.7	17.9	5.2	1.2	2.10	2.20	3.55	2.3	.03
3	1.2	.6	49.2	16.2	19.9	5.5	1.4	2.00	2.35	3.55	2.7	.04
4	1.7	.7	39.1	31.3	17.6	5.2	.9	.35	1.92	1.61	1.5	.08
5	3.0	2.3	47.2	19.1	19.3	5.2	1.3	1.22	2.38	2.79	1.9	.11
6	4.1	2.4	46.7	16.4	20.3	6.4	1.1	.50	2.00	1.82	2.1	.64
7	3.8	3.5	40.4	29.5	20.0	1.7	1.2	.35	2.17	1.78	1.0	.01
8	4.1	3.5	47.1	16.3	22.2	4.5	1.1	1.00	2.50	2.64	1.2	.07
9	4.2	3.9	36.5	27.1	19.7	7.8	1.3	.75	1.28	1.59	1.4	.07
10	4.0	3.9	38.5	25.4	17.9	8.3	1.0	.40	2.00	1.72	2.3	.13
11	4.2	3.9	48.3	18.0	23.9	1.8	1.1	.40	2.00	1.72	.5	.01
12	5.0	4.5	51.9	12.3	23.7	2.3	1.1	.40	2.20	1.85	.4	.02
13	4.7	4.8	45.3	19.3	26.1	1.6	.8	.22	1.80	1.40	.3	.03
14	5.4	5.0	41.2	20.6	22.1	6.0	1.2	1.00	1.42	1.93	.9	.04
15	5.7	5.5	38.9	24.1	20.7	6.4	1.2	.83	1.28	1.67	1.0	.10
16	7.5	6.1	44.8	11.2	18.4	11.6	1.1	.92	2.22	2.38	2.0	.10
17	6.9	6.4	42.2	19.4	22.4	4.3	1.2	.67	1.82	1.87	1.0	.05
18	7.1	6.9	49.2	8.7	26.7	3.6	.9	.58	2.38	2.15	.6	.03
19	7.4	6.9	45.3	16.9	26.0	1.4	.8	.25	2.00	1.57	.3	.01
20	8.6	7.8	48.2	17.7	16.1	5.2	1.2	.30	2.25	1.78	.3	.10
21	8.6	7.9	51.2	8.5	25.6	1.6	.9	.48	2.75	2.29	.3	.06
22	9.2	8.2	44.2	19.9	14.6	7.6	1.3	.32	1.97	1.62	1.0	.14
23	11.2	8.4	40.4	16.3	18.3	8.5	1.0	.70	1.62	1.77	1.7	1.26
24	9.2	8.9	45.3	18.6	22.4	1.5	.9	.67	1.42	1.60	.3	.03
25	12.0	8.9	44.6	12.6	20.0	6.2	1.2	.32	2.02	1.64	.9	1.27
26	9.2	9.1	48.9	8.8	28.3	1.2	.8	.52	2.20	1.97	.4	.02
27	12.5	10.5	41.6	14.2	19.0	6.8	1.2	.58	1.80	1.76	2.2	.51
28	11.2	10.6	46.7	12.9	24.7	1.1	.9	.50	2.05	1.85	.4	.02
29	11.6	11.2	38.5	18.8	23.5	3.2	1.0	.60	1.88	1.84	.6	.03
30	16.9	11.9	45.8	6.6	13.5	12.0	2.7	1.61	.50	1.94	.8	1.61
31	14.8	13.6	40.2	15.0	18.6	3.9	1.0	1.15	2.98	3.11	2.2	.04
32	14.5	14.3	44.1	10.9	25.5	1.7	.7	.47	2.00	1.79	.2	.04
33	16.4	14.8	32.7	28.0	17.1	1.8	1.1	.25	1.73	1.39	1.3	.03
34	18.0	15.6	40.1	15.2	17.5	3.5	1.1	.70	1.80	1.88	1.8	.15
Minimum	1.0	.2	32.7	6.6	13.5	1.1	.7	.22	.50	1.39	.2	.01
Maximum	18.0	15.6	51.9	31.3	28.3	12.0	2.7	2.10	2.98	3.55	2.8	1.61

---

[4] Fly ashes 1, 3, 14, and 29 are identified as A, B, X, and Y, respectively, in the article *Use of fly ash in concrete*, which appears in this issue.

used in sufficient amount. Because of variations in the behavior of different fly ashes, the amount to be used with a reactive combination of materials should be determined by testing concrete composed of those materials and the particular fly ash in question.

Table 2.—Specific gravity and fineness of fly ash samples

Fly ash number	Apparent specific gravity	Fineness of fly ash samples				
		Amount passing—		Air permeability, specific surface	Hydrometer method	
		No. 200 sieve	No. 325 sieve		Specific surface	Amount smaller than 0.03 mm.
		Percent	*Percent*	*Cm.²/gm.*	*Cm.²/gm.*	*Percent*
1	2.49	96.8	93.2	3,075	4,050	76
2	2.49	95.0	92.1	3,295	5,105	79
3	2.52	98.6	95.2	4,305	5,570	86
4	2.69	94.6	90.0	2,550	3,130	67
5	2.46	97.4	95.3	3,585	3,115	87
6	2.38	96.2	92.1	3,960	3,085	74
7	2.62	94.4	91.7	3,775	3,290	76
8	2.44	96.8	93.9	4,290	3,520	81
9	2.65	95.6	92.9	3,110	2,475	75
10	2.58	95.4	91.7	4,250	4,070	81
11	2.36	88.2	83.2	3,030	1,390	47
12	2.37	93.2	88.9	3,580	3,715	71
13	2.32	82.0	75.8	2,960	2,025	48
14	2.51	91.8	84.5	2,565	2,770	59
15	2.50	85.8	81.3	2,845	930	50
16	2.39	95.2	93.2	5,355	2,080	94
17	2.52	94.2	91.5	4,020	2,330	56
18	2.31	92.4	89.3	4,060	2,565	68
19	2.33	90.2	86.4	3,830	2,450	61
20	2.40	90.8	81.6	2,560	-----	--
21	2.12	57.2	56.2	2,430	2,155	31
22	2.39	90.4	82.0	2,985	2,088	67
23	2.30	85.4	76.6	3,315	1,260	67
24	2.41	92.2	86.6	3,140	1,970	60
25	2.30	77.0	74.8	4,115	2,225	47
26	2.22	87.6	82.8	4,405	3,255	68
27	2.31	82.8	82.5	4,650	-----	--
28	2.24	85.0	81.9	4,295	1,965	50
29	2.43	93.8	89.4	3,220	2,000	66
30	2.20	33.1	----	-----	-----	--
31	2.42	96.0	93.2	4,795	2,435	79
32	2.25	87.2	80.9	4,400	2,250	51
33	2.44	81.5	76.1	3,860	1,795	38
34	2.29	81.0	78.1	4,610	2,230	62

Chemical and Physical Properties of Fly Ash Samples

The results of chemical analyses showing the principal constituents of the samples of fly ash used in this investigation are given in table 1. In recognition of the importance of the carbon content of the fly ash, identification numbers were assigned to the samples on the basis of increasing amounts of carbon.

The carbon in fly ash represents unburned portions of the coal and varies with the conditions of burning at the plant. Some fly ashes are produced which contain more carbon than any of the samples listed in table 1, but such materials are not likely to be considered for use in concrete. Although the loss on ignition of fly ash is generally considered to be closely related to the carbon content, table 1 and figure 2 show that the loss can be considerably greater in some cases. Of these samples, five showed differences equal to or greater than 2.0 percent—the greatest difference being 5.0 percent for sample 30. These large differences usually occur when there is an appreciable amount of calcium hydroxide or carbonate present. At the ignition temperature used, 600° C., these materials decompose to the oxide and either water or carbon dioxide. As the latter two constituents are volatile they increase the loss on ignition. Combined moisture from other minerals may also increase the loss.

It should be noted that the practice of limiting the carbon content by specifying a maximum, limit for ignition loss will insure, for all practical purposes, that the carbon content will be less than this limit and sometimes by a significant amount. The one exception to this general statement is the case of sample 13 where the carbon content exceeds the loss on ignition by 0.1 of a percentage point. This small difference is within the limits of experimental error.

It is possible that some of the samples tested contain small amounts of manganese, titanium, and possibly phosphorus. The titanium and phosphorus, if present, are included with the reported percentage of alumina (Al_2O_3). When manganese is present, most of it is included with the magnesia (MgO) and the balance with the lime (CaO). This manner of determination and reporting follows the usual practice in silicate analysis. It is believed that these minor constituents have no significance in this investigation. Details of the methods followed in making the chemical analyses are given on page 139 in procedure 1 of Appendix A.

The specific gravity and fineness for all samples of fly ash are given in table 2. Sample 30 had only 33.1 percent passing the No. 200

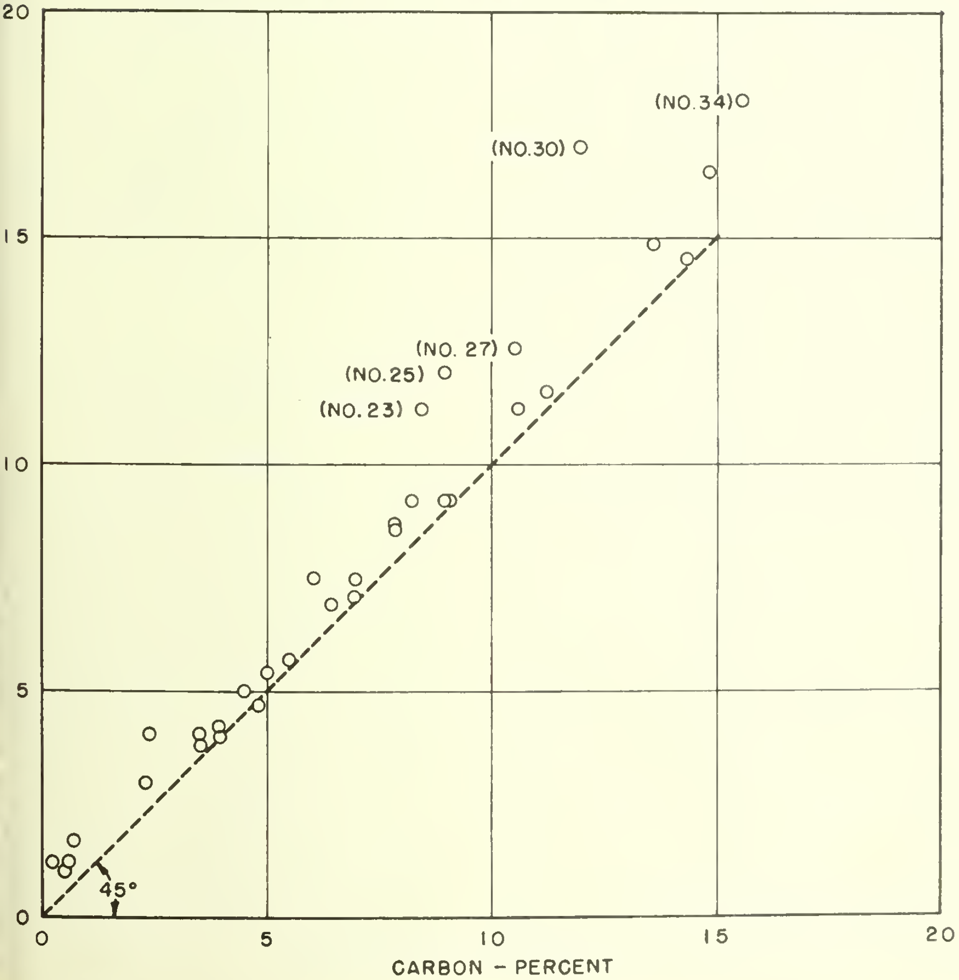

Figure 2 (Left).—Relation between carbon content and loss on ignition for 33 fly ashes.

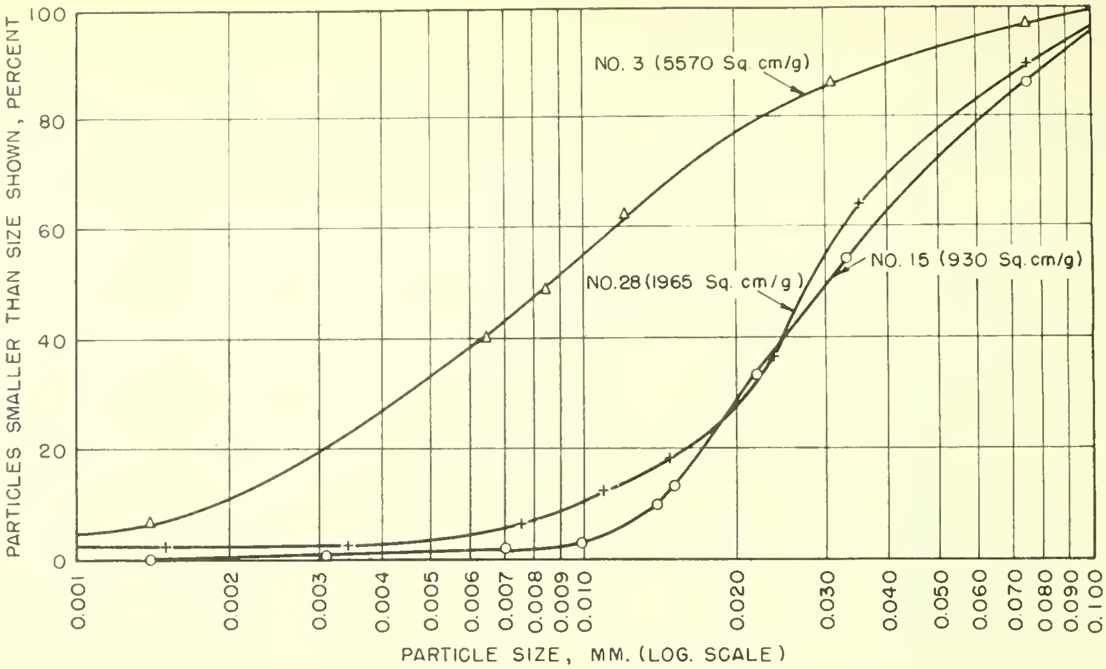

Figure 3.—Grain-diameter accumulation curves for 3 fly ashes.

sieve, and mortar tests were not made with this fly ash. The specific gravities of the fly ashes vary from 2.12 to 2.69 and are found to bear a general relation to the amounts of iron oxide present in the sample—high specific gravities being indicative of high iron contents.

The fineness of the fly ash was determined by three methods: wet sieving through the Nos. 200 and 325 sieves, air permeability, and hydrometer analysis. The amount passing the No. 325 sieve was determined in accordance with the procedure outlined in section 12 of the test for Fineness of Portland Cement by the Turbidimeter, ASTM Designation C 115–53. Specific surface by air permeability was determined by the test for Fineness of Portland Cement by Air Permeability Apparatus, ASTM Designation C 204–55. The weight of sample used in this test was determined by trial as the weight necessary to obtain a hard, firm bed in the permeability cell. For many samples it was found necessary to use a porosity for the test bed of fly ash which differed considerably from the porosity used in the calibration of the apparatus. Fineness by the hydrometer method was determined in accordance with the Standard Methods of Mechanical Analysis of Soils, AASHO Designation T 88–54, using 125 ml. of a 0.4N solution of sodium hexametaphosphate as the deflocculating agent. Dispersion cup A shown in figure 3 of the AASHO method was used. The initial weight of each sample was corrected for the amount of water-soluble material present in calculating the amount of sample smaller than each size.

The grain-diameter accumulation curve obtained by the hydrometer analysis was used to calculate the specific surface based on the amounts of the sample estimated to have average diameters of 0.7, 2, 3, 4, 5, 6, 7, 8, 9, 12, 20, 30, 40, 50, and 60 microns. The grain-diameter accumulation curves for three fly ashes are shown in figure 3. The curves for the remaining samples used in this investigation lie essentially within the area bounded by the curves for fly ashes Nos. 3 and 15, which have the highest and lowest specific surfaces, respectively, as calculated by this method. The curve for sample 28 is included to illustrate the discrepancy that is often shown between values for specific surface obtained by the air permeability and hydrometer methods.

The specific surfaces by the air permeability method for fly ashes Nos. 3 and 28 were about 4,300 sq. cm. per gram, which classifies both as relatively fine materials. The grain-diameter accumulation curves, however, are widely different. The specific surfaces for these two materials are also quite different when determined by the hydrometer method, being 5,570 sq. cm. per gram for sample No. 3 and 1,965 sq. cm. per gram for sample No. 28.

In table 3, average values for fineness as determined by each method are shown for the samples of fly ash which have been grouped with respect to carbon content. Samples 20, 27, and 30 are not included as these materials were not tested by all three methods for fineness. For the samples considered here, those with the most carbon were indicated to be the finest by the air permeability method.

Table 3.—Average fineness of fly ash samples by groups

Range of carbon content	Number of samples	Average fineness values			
		Amount passing No. 325 sieve	Air permeability, specific surface	Hydrometer method	
				Specific surface	Amount smaller than 0.03 mm.
Percent		*Percent*	*Cm.²/gm.*	*Cm.²/gm.*	*Percent*
0–4	11	91.9	3,570	3,530	75
4–8	9	83.0	3,520	2,340	60
8–12	7	82.0	3,640	2,110	61
12–15.6	4	81.0	4,420	2,180	58

Determinations of fineness by both the No. 325 sieve and the hydrometer showed, however, that the finest fly ashes were those containing relatively little carbon.

These data indicate that the determination of the fineness of fly ash by the air permeability method is likely to furnish misleading information. A possible cause of this can be seen by examining the photomicrographs of fly ash in figure 1. The transparent, spherical particles are composed chiefly of the inorganic portion of each fly ash. The dark, irregularly shaped particles present in fly ash No. 15 are composed of carbon and appear to be highly porous. It is believed that the results of the air permeability method are influenced by both the external and internal surfaces of particles of carbon. Consequently, an increase in the carbon content of fly ash may be accompanied by an increase in total surface area even though the external surface area and the actual fineness of the material decrease.

Compressive Strength Tests of Mortar Containing Fly Ash

The test specimens for determining the effect of fly ash on the compressive strength o mortar were 2-inch cubes, and were made both from a control mortar containing no fly ash and test mortars in which various amounts o the same cement were replaced with fly ash o a solid volume basis. Each fly ash was used t replace 10, 20, 35, and 50 percent of th cement in a 1:2.75 mix by weight using graded Ottawa sand. Control specimens were pre pared each day that specimens containing fl ash were made. All cement-fly ash combina tions were repeated with each of three cements the properties of which are given in table 4

The mixing of the mortars and molding o the specimens were performed in accordanc with the test for Compressive Strength o Hydraulic Cement Mortars, ASTM Designa tion C 109–49. As specified in this procedure a uniform consistency of all mortars was main

Table 4.—Properties of cements used in preparing mortars containing fly ash

	Cement A	Cement B	Cement C
Chemical analysis (in percent):			
Silicon dioxide (SiO_2)	21.8	22.3	21.7
Aluminum oxide (Al_2O_3)	5.7	5.4	6.1
Ferric oxide (Fe_2O_3)	2.5	2.4	2.8
Calcium oxide (CaO)	61.9	66.1	64.9
Magnesium oxide (MgO)	2.6	1.0	1.2
Sulfur trioxide (SO_3)	1.9	1.7	1.6
Loss on ignition	2.1	1.2	1.2
Insoluble residue	.19	.12	------
Sodium oxide (Na_2O)	.33	.04	.13
Potassium oxide (K_2O)	.86	.15	.72
Equivalent alkali as Na_2O	.90	.13	.60
Compound composition (in percent):			
Tricalcium silicate (C_3S)	39	55	50
Dicalcium silicate (C_2S)	33	22	25
Tricalcium aluminate (C_3A)	11	10	12
Physical Properties:			
Fineness (Wagner turbidimeter) cm²/gm	1,785	1,625	1,875
Normal consistency percent	24.5	25.0	25.5
Air content of mortar do	10.0	6.8	7.8
Compressive strength[1]:			
At 7 days p. s. i	2,260	2,960	4,325
At 28 days do	3,560	4,725	6,000
Time of set:			
Initial hours	2.6	4.2	4.0
Final do	4.0	6.4	5.1

[1] ASTM Method C 109.

ained by adjusting the amount of water added o each mix. The effect of a particular fly ash n the amount of mixing water required can e indicated conveniently by calculating the atio of the amount of water required by the ement-fly ash mortar to that required by the ontrol mortar. This ratio will be referred to ubsequently as "water requirement ratio." fter removal from the molds at an age of 24 ours, all specimens were stored at 73° F. in noist air in tightly covered containers until ested at ages of 28, 91, and 365 days.

The water requirement ratios used for prearing the mortars containing fly ash are given n table 5. The results of the tests for comressive strength of these mortars are given in ables 6–8 (pp. 128–130). For the convenince of the reader, the actual average strengths btained in tests of the control mortars (withut fly ash) are given, but the strengths of the ortars containing fly ash are reported as atios (percent) of the strength of the control ortar. By this means, direct comparisons an be made to study the effect of any variable ncluded in these tests. Figures 4 and 5 how averages of the compressive strength atios obtained with the three cements for ach fly ash.

As shown in tables 6–8 and figures 4–5, an ncrease in age of test is usually accompanied y an increase in strength ratio. This is asumed to indicate that each fly ash tested has ome pozzolanic properties. The few results hich do not follow this general trend are elieved to be instances where the effects of ozzolanic action have been overshadowed by ther experimental variations.

The replacement of part of the cement by y ash usually resulted in some loss of strength t an age of 28 days, since at this age sufficient ozzolanic action had not taken place to comensate for the reduction in strength caused by e use of less cement. This early loss in rength became greater as the amount of replacement of cement by fly ash was increased. At greater ages, however, some fly ashes furnished sufficient pozzolanic activity to overcome this early deficiency in strength. The general decrease in strength ratio with increase in the carbon content of the fly ash is also of interest, as well as the individual fly ashes which gave results departing from this trend.

Effect of Richness of Mix

As shown in tables 6–8, four fly ashes, Nos. 1, 3, 14, and 29, were tested in 1:2, 1:2.75, and 1:3.5 mortars. The average results of tests of these mortars obtained with a 35-percent replacement of the three cements are shown in figure 6 (p. 131). For three of the four fly ashes, the richest mix showed the least reduction in strength ratio at an age of 28 days and the leanest mix showed the most. For the fourth fly ash, the strength ratios at 28 days were practically the same. At an age of 1 year, however, the leanest mix showed the greatest strength ratio for all fly ashes. This shows that ultimately fly ash may be of most value in the leaner mortars and concretes. Similar results have been found with the use of various pozzolans in concrete (*5*).

Influence of Cement Characteristics

The characteristics of the cements used had a marked effect on the strength ratio of mortar containing fly ash. The effect of differences in cement is illustrated by the graphs of figure 7 (p. 131) which show the age-strength relations obtained for a 35-percent replacement with each of three fly ashes of different carbon content and also the average of the strength ratios obtained with 33 fly ashes The use of cement A produced appreciably higher strength ratios than cement B with all fly ashes. With few exceptions, mortars prepared with cement C produced strength ratios intermediate between those shown for mortars containing cements A or B.

To investigate further the variations in strength ratios which might occur because of differences in cements, compressive strength specimens were prepared with 10 cements other than those previously used. In each case, a 1:2 mortar was prepared with fly ash

Table 5.—Water requirement ratio of cement-fly ash mortars

Fly ash number	Water requirement ratio based on control mortar											
	10-percent replacement			20-percent replacement			35-percent replacement			50-percent replacement		
	Cement A	Cement B	Cement C	Cement A	Cement B	Cement C	Cement A	Cement B	Cement C	Cement A	Cement B	Cement C
1:2.75 Mortar												
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
1	96	94	94	93	93	92	90	90	89	86	86	85
2	100	100	101	97	96	99	92	91	94	88	87	89
3	96	94	94	93	93	92	90	90	89	87	87	85
4	103	101	101	97	99	99	92	97	97	91	96	95
5	101	100	101	97	97	97	94	94	93	90	91	90
6	101	101	101	99	99	99	97	100	97	101	101	101
7	104	103	103	194	103	102	103	101	100	103	103	100
8	101	101	102	100	101	100	97	99	97	94	94	95
9	100	101	102	100	101	102	99	100	100	97	99	98
10	100	99	102	100	100	102	100	101	103	100	101	103
11	104	103	103	101	104	101	104	101	101	103	101	103
12	101	100	102	98	100	190	97	99	98	94	97	97
13	100	100	102	101	103	105	103	104	106	104	109	111
14	101	100	100	101	100	100	101	100	100	103	100	101
15	101	103	103	104	104	105	106	107	106	107	109	109
16	103	100	101	100	97	99	97	94	96	94	93	94
17	98	100	101	97	101	100	97	101	101	97	101	101
18	102	99	103	102	100	101	102	100	101	102	100	101
19	98	103	103	98	103	105	98	103	105	102	106	108
20	100	100	100	98	101	100	98	100	101	98	101	101
21	102	100	101	106	106	108	112	113	116	118	122	125
22	102	103	103	103	106	106	108	110	111	111	113	124
23	100	101	100	100	103	101	102	106	104	105	109	107
24	100	100	98	102	103	101	103	104	103	106	106	106
25	101	101	101	103	104	104	109	109	109	116	115	116
26	100	103	103	102	107	108	105	109	112	109	113	116
27	107	107	107	109	109	109	118	118	115	125	125	119
28	104	104	101	106	104	106	110	109	109	118	115	115
29	101	101	101	103	103	103	106	103	104	108	106	106
31	100	99	99	99	99	97	97	99	97	99	100	99
32	103	103	101	104	106	104	109	109	109	113	113	113
33	101	104	101	106	109	107	113	119	117	122	126	125
34	109	109	109	118	116	116	125	125	125	134	134	137
1:2 Mortar												
1	99	97	97	94	93	93	89	89	89	84	84	85
3	96	99	99	92	93	94	86	90	92	82	86	87
14	100	101	100	102	100	100	101	100	100	101	100	100
29	100	101	100	100	103	101	105	106	104	110	109	107
1:3.5 Mortar												
1	99	97	97	95	95	93	91	92	91	88	89	88
3	97	97	96	95	95	93	91	92	91	88	89	87
14	101	97	100	101	99	100	101	99	100	101	100	100
29	101	101	101	104	103	103	107	105	104	108	107	105

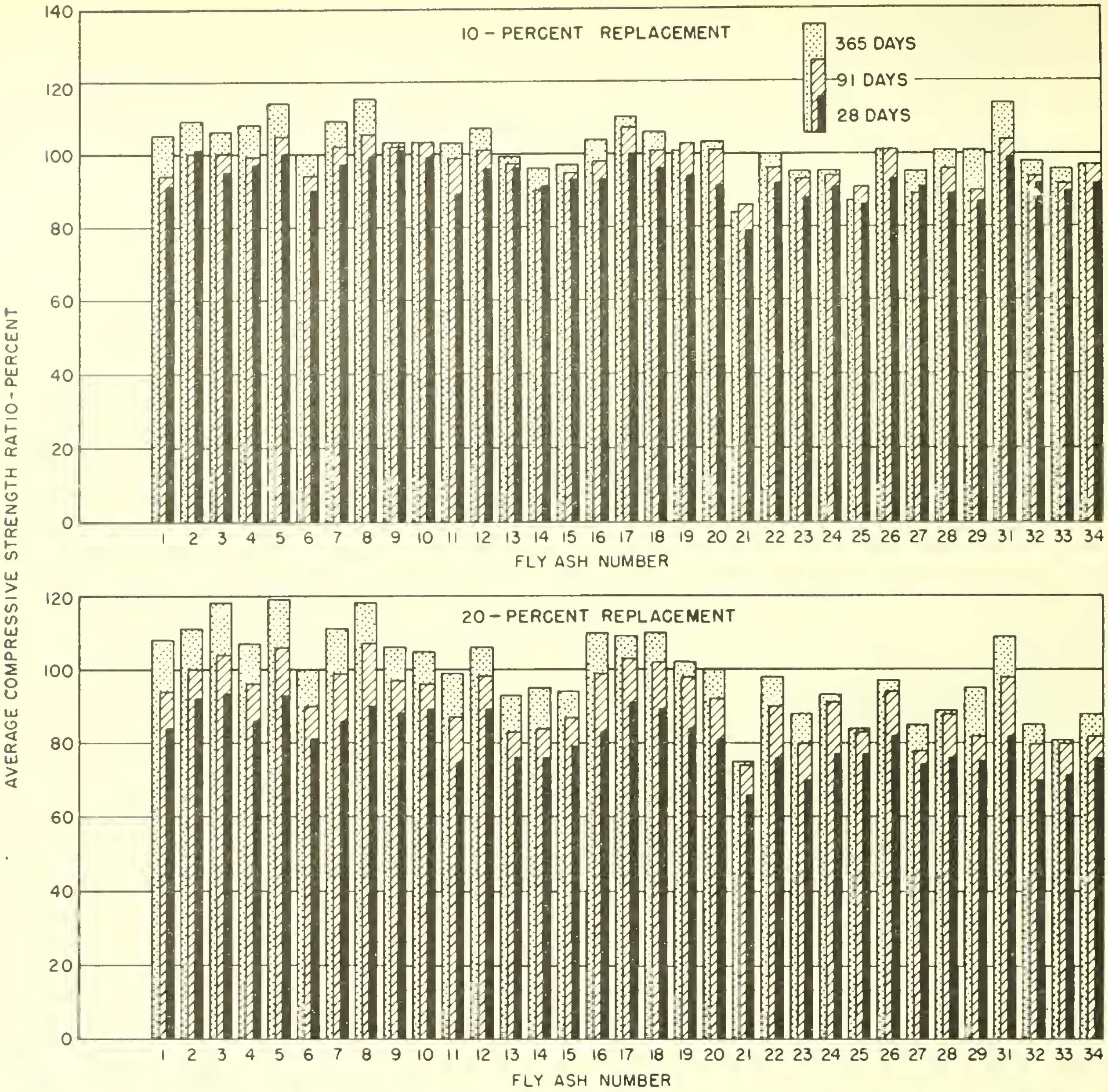

Figure 4.—Development of compressive strength in cement-fly ash mortar (1:2.75 mix, 10- and 20-percent replacement of cement with fly ash).

No. 3 replacing 50 percent of the cement. The cements were selected to provide a wide range in alkali content, tricalcium silicate content, and mortar strength. Cements A, B, and C also varied considerably in these properties. The results obtained with the 10 cements are shown in table 9 (p. 132). As in the previous tests, a wide range in compressive strength ratios was found for the single fly ash used.

The samples of cement in this table have been separated into three groups on the basis of their alkali content. As shown in figure 8 (p. 132) the rate of gain in strength ratio for the groups of mortars prepared with high, medium, and low alkali cements was influenced by the alkali content of the cements. The mortars prepared with high alkali cement reached maximum strength ratios at some age between 28 and 91 days. Those prepared with medium alkali cement required about 91 days to gain a maximum strength ratio. The mortars prepared with the cements of the low alkali group appeared to be still gaining in strength ratio when the test was terminated at an age of one year.

Davis (*5*) has stated that a larger replacement of cement by a pozzolan may be made when type I or II cement is used than when type IV cement is used. Types I and II cements contain more tricalcium silicate than type IV and should liberate more lime which can combine with the pozzolan. To check this hypothesis, the data given in table 9 were rearranged on the basis of the tricalcium silicate content of the cements and placed in groups in which the average tricalcium silicate content of the cement was 31, 45, and 60 percent. The age-strength data for these groups are shown in figure 9 (p. 132). At an age of one year, when the strength resulting from pozzolanic action would be more fully developed for all cements than at early ages, the strength ratios of the three groups of cement vary directly with the tricalcium silicate content. At ages less than one year, no definite relation between strength and trica cium silicate is apparent.

The data given in table 9 and figures 8- indicate that the cement used has a marke effect on the development of strength fro pozzolanic action in a cement-fly ash morta At the earlier ages of test, the alkali in th cement appears to accelerate the reactio between the cement and the fly ash. Th agrees with data reported by Alexander (*6* At the greater ages of test, the type of ceme or more definitely, the amount of tricalciu silicate in the cement appears to govern t benefits derived by the addition of fly as to the mortar.

Pozzolanic Strength Index

To determine the relations existing betwee the strength of cement-fly ash mortars an the properties of fly ash, it was necessary t select one standard of comparison for t strength of the mortar. The strength rati

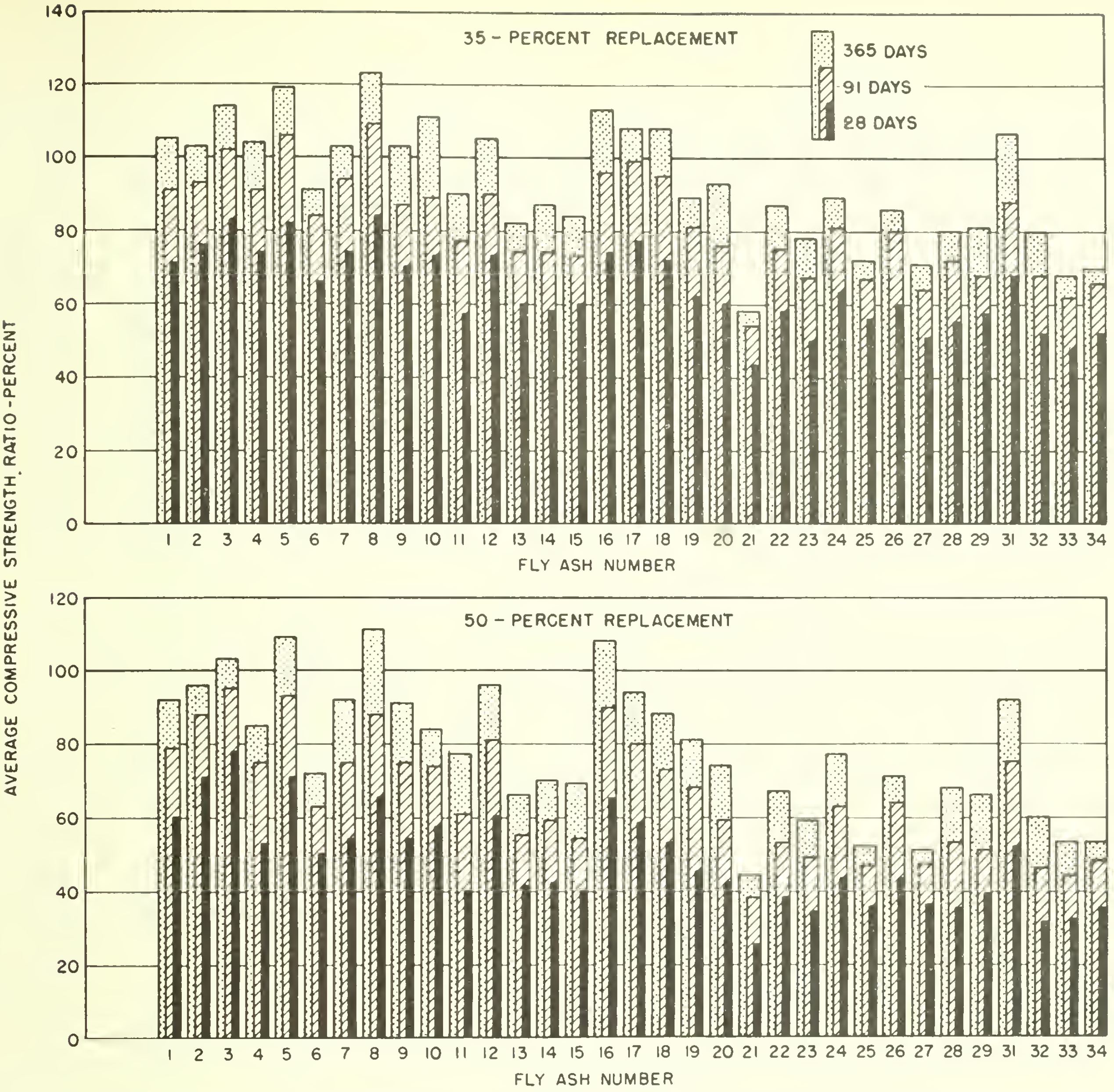

Figure 5.—Development of compressive strength in cement-fly ash mortar (1:2.75 mix, 35- and 50-percent replacement of cement with fly ash).

of mortar obtained at an age of 28 days with 35-percent replacement of cement by fly ash were considered the most suitable for this purpose. A large percentage of replacement, such as this amount, was found to show the effect of different fly ashes on the strength of mortar to the most marked extent. A 35-percent replacement was also desirable as it approximated the maximum amount of fly ash generally used in concrete. To obtain the most dependable value for assessing the quality of each fly ash, it appeared desirable to average the results for the three cements used. The average of the strength ratios obtained for each fly ash at an age of 28 days with 35-percent replacement of each of the three cements will be referred to in the remainder of this article as the "pozzolanic strength index."

The pozzolanic strength indexes of the 33 fly ashes are plotted in figure 10 (p. 133) together with averages of the strength ratios obtained for the same mortars at 91 and 365 days. The samples are arranged from left to right in decreasing order of their pozzolanic strength index. If the average strength ratios for 91 or 365 days were used for rating the fly ashes, it can be seen that the order for each age would be somewhat different from that shown. However, the 28-day results should have most significance regarding the use of fly ash in concrete for highways, since this is the age at which final strength tests are customarily made and at which the concrete may be placed in service.

It should be noted, moreover, that a high pozzolanic strength index generally indicates a continued high level of strength development. To illustrate, the 13 fly ashes having pozzolanic strength indexes of 70 or greater showed an average increase in strength ratio of 33.7 percent from 28 to 365 days, whereas the 20 fly ashes having indexes of less than 70 showed an average increase of only 25.4 percent during the same period. All of the 13 fly ashes in the group having the higher pozzolanic strength indexes showed increases in strength ratio greater than 25.4 percent, but only one fly ash of the lower index group had an increase in strength ratio of over 33.7 percent. It appears that there is a marked difference in quality between the two groups.

Relation Between Fineness of Fly Ash and Pozzolanic Strength Index

A high degree of fineness is generally desired for any pozzolanic material used as an admixture in concrete. In this series of tests, the fineness of nearly all fly ashes was determined by three methods: air permeability, hydrometer, and wet sieving through the Nos. 200 and 325 sieves. The results obtained are shown in table 2 and are also plotted against the pozzolanic strength index in figure 11 (p. 133).

No significant relation was found between the specific surface as determined by the air permeability or hydrometer methods and the pozzolanic strength index. Even for fly

Table 6.—Compressive strength of cement-fly ash mortars prepared with cement A [1]

Fly ash number	Compressive strength of control mortar (without fly ash) [2]			Compressive strength ratio of mortars containing fly ash [3]											
				10-percent replacement			20-percent replacement			35-percent replacement			50-percent replacement		
	28 days	91 days	365 days	28 days	91 days	365 days	28 days	91 days	365 days	28 days	91 days	365 days	28 days	91 days	365 days
1:2.75 Mortar															
	P. s. i.	P. s. i.	P. s. i.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.
1	3,180	3,840	4,240	92	94	113	90	100	121	82	101	110	72	84	90
2	3,760	4,440	4,940	100	98	112	95	104	117	84	100	106	80	94	98
3	3,180	3,840	4,240	97	99	111	98	110	126	97	116	123	95	105	111
4	3,420	3,770	4,440	101	108	118	96	106	119	82	103	112	64	93	90
5	3,000	3,630	4,120	112	117	121	102	117	124	102	127	135	93	112	115
6	3,360	3,780	4,290	96	100	101	91	99	113	82	94	103	67	76	79
7	3,420	3,770	4,440	108	115	118	99	116	126	86	112	113	65	89	95
8	3,420	3,770	4,440	107	115	125	101	120	130	96	126	131	87	114	122
9	3,760	4,440	4,940	97	100	102	86	97	108	76	98	111	62	89	102
10	3,540	4,120	4,710	96	99	97	95	99	111	83	99	108	72	82	87
11	3,440	4,110	4,400	96	101	110	82	90	105	65	87	103	51	74	87
12	3,680	4,170	4,590	94	103	112	88	106	113	83	105	112	67	93	97
13	3,540	4,120	4,710	95	96	99	75	84	90	64	84	88	45	61	71
14	3,330	3,940	4,550	90	93	99	75	86	94	60	77	89	47	68	77
15	3,760	4,440	4,940	89	93	93	81	89	95	67	80	89	51	68	77
16	3,440	4,110	4,400	103	102	112	95	114	127	95	121	138	91	114	124
17	3,520	3,830	4,580	104	113	112	96	107	108	87	117	118	75	97	102
18	3,740	4,450	4,750	95	100	103	90	103	115	86	108	121	64	82	105
19	3,520	3,830	4,580	95	105	98	88	108	108	72	96	100	58	86	96
20	3,740	4,450	4,750	94	101	101	85	96	108	66	81	103	45	63	78
21	3,740	4,450	4,750	78	82	84	72	75	81	53	60	66	30	44	48
22	3,520	3,830	4,580	96	99	99	86	100	105	68	88	95	46	67	73
23	3,800	4,020	4,490	88	96	98	75	88	96	54	75	85	42	56	65
24	3,520	3,830	4,580	92	100	100	88	101	98	71	96	94	56	81	88
25	3,360	3,780	4,290	92	93	93	84	90	89	62	74	76	42	52	52
26	3,520	3,830	4,580	94	108	107	92	109	110	69	93	94	54	81	84
27	3,000	3,630	4,120	98	94	97	83	85	89	62	76	82	41	51	52
28	3,440	4,110	4,400	93	101	111	86	98	105	66	85	95	44	65	81
29	3,330	3,940	4,550	84	91	95	73	81	93	60	76	84	46	60	74
31	3,520	3,990	4,560	108	111	114	91	109	116	80	109	115	70	99	105
32	3,520	3,990	4,560	95	101	98	78	89	93	64	83	92	39	59	74
33	3,520	3,990	4,560	94	90	93	81	91	89	60	76	79	41	56	64
34	3,000	3,630	4,120	93	100	97	85	93	92	67	82	80	44	56	58
Average	------	------	------	96	101	104	87	99	106	74	94	102	59	78	85
1:2 Mortar															
1	4,670	5,340	5,890	102	102	113	99	108	124	97	115	119	92	104	114
3	4,560	5,490	6,510	102	103	108	103	115	115	109	121	114	114	117	109
14	4,960	5,800	6,840	96	102	104	80	87	94	63	80	86	54	73	78
29	4,960	5,800	6,840	95	104	102	75	92	100	64	81	91	44	64	75
Average	------	------	------	99	103	107	89	100	108	83	99	102	76	90	94
1:3.5 Mortar															
1	2,540	2,950	3,170	97	108	117	83	97	124	73	98	121	61	88	103
3	2,540	2,950	3,170	92	100	113	92	107	133	91	110	136	82	103	117
14	2,540	2,950	3,170	88	97	103	78	67	76	57	82	108	42	68	98
29	2,330	2,920	3,000	105	106	133	95	97	122	72	85	109	51	61	88
Average	------	------	------	96	103	116	87	92	114	73	94	118	59	80	102

[1] Moist air storage at 73° F.
[2] Each value is an average of three 2-inch cubes.
[3] Reported as a percentage of the strength of control mortar.

ashes having low carbon contents (5.0 percent or less), no definite correlation was found between strength and specific surface. The amount of material passing the No. 325 sieve, however, showed a reasonably definite relation to the pozzolanic strength index. In view of the results obtained with the No. 325 sieve, data from the hydrometer determinations were used to obtain values for amounts of each sample finer than the 0.03, 0.025, 0.02, and 0.01 mm. sizes. Correlation with the pozzolanic strength index was found to be best when the size finer than 0.03 mm. was used, but became poorer with the smaller sizes in the group.

Relation Between Carbon and the Pozzolanic Strength Index

Minnick (7) has shown that the strengths of fly ash-cement mortars are adversely affected by fly ashes having high carbon contents. In general, the data shown in tables 6–8 support these findings. A similar trend is evident in figure 12 (p. 134) in which the relation between the pozzolanic strength index and carbon content is shown. The scattering of points in this figure indicates the effect of variables other than carbon. For example, fly ashes 21 and 31 appear to have properties which cause them to deviate significantly from the general trend. The rather low strength index value of sample 21 may be partly attributed to an unusual coarseness of the fly ash and high water requirement of the mortar prepared with this material. Although sample 31 contained 13.6 percent carbon, its high fineness and low water requirement probably account for the high strength index.

The effect of carbon content on the strength of mortar was investigated in a special series of tests in which five selected fly ashes having carbon contents ranging from 0.2 to 14.3 percent were tested before and after heating to a temperature of 550° C. This temperature is considered sufficient to remove all the carbon, and although some oxidation of ferrous iron to the ferric state may have occurred, it is believed that the pozzolanic properties of the inorganic portion of the fly ash were unaltered.

Strength tests were conducted on control specimens prepared from a 1:2.75 mortar and test specimens in which a portion of the cement was replaced with fly ash by three different methods as follows:

1. Thirty-five percent of the solid volume of cement used in the control mortar was replaced by an equal volume of fly ash as received.
2. Thirty-five percent of the solid volume of cement used in the control mortar was

Table 7.—Compressive strength of cement-fly ash mortars prepared with cement B [1]

Fly ash number	Compressive strength of control mortar (without fly ash) [2]			Compressive strength ratio of mortars containing fly ash [3]											
				10-percent replacement			20-percent replacement			35-percent replacement			50-percent replacement		
	28 days	91 days	365 days	28 days	91 days	365 days	28 days	91 days	365 days	28 days	91 days	365 days	28 days	91 days	365 days
1:2.75 MORTAR															
	P. s. i.	*P. s. i.*	*P. s. i.*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1	5,130	6,260	7,030	92	92	98	82	93	102	66	84	97	51	75	87
2	5,020	6,360	6,560	108	105	113	92	97	107	67	85	97	63	79	87
3	5,130	6,260	7,030	95	103	105	93	101	113	75	93	107	69	89	98
4	4,840	6,000	6,440	99	93	103	88	95	103	72	89	105	46	63	76
5	5,150	6,190	6,780	94	100	108	88	97	113	69	91	111	55	76	98
6	5,560	6,730	6,930	83	83	94	70	76	88	54	76	83	35	50	66
7	5,290	6,240	6,390	90	92	99	78	88	101	65	84	94	45	67	92
8	5,290	6,240	6,390	88	95	104	81	94	107	74	97	121	45	70	102
9	4,840	6,000	6,440	99	101	105	81	89	104	56	70	92	46	68	87
10	4,840	6,000	6,440	105	107	109	87	93	105	66	82	97	46	67	77
11	5,220	6,410	7,040	83	91	91	67	80	90	48	65	77	33	50	64
12	5,020	6,360	6,560	97	99	101	80	82	94	63	78	98	53	72	93
13	4,840	6,000	6,440	99	100	101	77	86	94	51	66	76	35	50	60
14	4,930	6,290	6,620	92	86	95	75	79	93	55	69	85	36	51	62
15	4,840	6,000	6,440	91	98	102	72	81	93	48	63	80	28	40	62
16	5,220	6,410	7,040	87	95	97	74	86	96	57	78	96	47	72	92
17	4,760	5,540	5,870	92	100	108	85	101	114	65	86	101	43	64	86
18	5,030	6,040	6,540	87	95	100	79	90	104	58	78	95	40	61	82
19	4,910	5,770	6,260	87	100	104	79	89	95	58	74	86	37	51	71
20	4,760	5,540	5,870	86	97	107	75	85	100	54	70	88	38	52	73
21	5,030	6,040	6,540	73	81	80	58	67	69	33	45	51	20	33	41
22	4,760	5,540	5,870	86	94	103	65	78	91	51	68	85	31	45	66
23	4,840	5,900	6,590	89	92	93	68	73	81	47	60	72	29	42	51
24	4,840	5,900	6,590	97	93	98	72	90	92	59	73	82	37	51	67
25	5,560	6,730	6,930	75	82	72	71	75	80	49	57	65	26	39	49
26	4,840	5,900	6,590	94	98	98	76	88	90	54	74	83	37	56	64
27	5,150	6,190	6,780	85	78	90	66	71	84	43	56	65	28	40	47
28	5,220	6,410	7,040	87	92	97	70	81	85	44	61	69	26	41	55
29	4,930	6,290	6,620	87	89	100	73	77	91	53	60	75	33	44	57
31	5,220	6,640	6,930	92	95	106	77	91	105	60	71	96	37	51	73
32	5,220	6,640	6,930	84	85	93	63	72	78	44	57	70	25	36	48
33	5,220	6,640	6,930	83	88	93	64	73	79	40	53	61	23	34	44
34	5,150	6,190	6,780	92	92	96	68	70	87	41	55	62	28	41	49
Average				90	94	99	76	84	95	56	72	86	39	55	70
1:2 MORTAR															
1	8,440	10,390	10,970	87	93	99	82	89	96	70	85	99	60	71	81
3	8,560	10,260	10,960	94	97	108	87	100	108	82	95	107	76	83	91
14	8,440	10,390	10,970	90	91	101	71	82	92	56	64	80	40	49	62
29	8,440	10,390	10,970	92	93	101	76	80	89	53	60	74	33	43	57
Average				91	94	102	79	88	96	65	76	90	52	62	73
1:3.5 MORTAR															
1	3,260	4,270	4,500	85	98	113	71	90	103	58	79	100	42	65	94
3	3,260	4,270	4,500	86	93	107	77	88	106	63	78	107	52	71	97
14	3,260	4,270	4,500	75	84	93	60	70	87	43	60	74	28	44	64
29	3,320	4,140	4,540	86	90	96	71	79	90	51	64	78	31	43	53
Average				83	91	102	70	82	96	54	70	90	38	56	77

[1] Moist air storage at 73° F.
[2] Each value is an average of three 2-inch cubes.
[3] Reported as a percentage of strength of control mortar.

replaced by an equal volume of ignited fly ash. This replacement resulted in more inorganic material being added to the mix than by the first method.

3. Thirty-five percent of the solid volume of cement used in the control mortar was replaced by a smaller volume of ignited fly ash equal to that of the inorganic portion of the fly ash added by the first method. The replacement in this case did not result in a 1:2.75 mix but one in which the cement, sand, and inorganic constituents from the fly ash were the same as for the mortar in method 1.

The water requirement and strength ratios for this series are shown in table 10 (p. 135). The results obtained with fly ash No. 1 were essentially the same for the three methods of replacement, which indicates that the pozzolanic properties of this fly ash were not changed by ignition at 550° C. No effect would be expected to result from the removal of the 0.2 percent carbon originally present in this sample. An explanation of the strength ratios obtained with fly ash No. 7 is not apparent.

For the remaining three samples, the strength ratios were highest when replacement was made by method 2 which provided the greatest amount of carbon-free fly ash, and lowest by method 1 where carbon was present in the fly ash. The direct effect of the carbon is found by comparing the results obtained by methods 1 and 3 since the amount of the inorganic constituent is the same in either case. The slightly lower strengths obtained by method 1 where carbon was present can be attributed to the higher water requirements of these mortars. The higher strengths obtained by method 2 as compared with method 3 must result from the greater quantity of carbon-free ash added by that method since no carbon is present in either case. The range in strength ratios at 28 days from 71 to 87 percent which was found for the five fly ashes tested by method 2 also indicates that essential differences exist between the fly ashes which cannot be attributed to their carbon content.

These results indicate that the carbon in fly ash has at least two adverse effects on the strength of mortar. Some reduction in strength occurs as a result of the increased water requirement of the mortar caused by the presence of carbon. A reduction in strength also appears to result from the fact that the amount of beneficial inorganic material added to the mix is reduced by the amount of carbon present.

Relation of Water Requirement to Pozzolanic Strength Index

In the preparation of the mortars for the compressive strength tests, it was found that

Table 8.—Compressive strength of cement-fly ash mortars prepared with cement C [1]

Fly ash number	Compressive strength of control mortar (without fly ash) [2]			Compressive strength ratio of mortars containing fly ash [3]											
				10-percent replacement			20-percent replacement			35-percent replacement			50-percent replacement		
	28 days	91 days	365 days	28 days	91 days	365 days	28 days	91 days	365 days	28 days	91 days	365 days	28 days	91 days	365 days
1 : 2.75 MORTAR															
	P. s. i.	*P. s. i.*	*P. s. i.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
1	5,920	6,240	6,590	90	96	103	79	89	102	64	87	107	58	79	98
2	5,390	5,670	6,070	96	97	101	89	99	108	76	94	106	69	90	104
3	5,920	6,240	6,590	92	99	102	89	102	114	76	98	113	70	91	101
4	5,490	5,870	5,860	90	96	103	75	87	99	67	82	96	50	68	89
5	5,670	5,740	5,770	94	99	114	89	104	119	74	100	117	66	90	13
6	5,250	5,270	5,530	92	100	104	81	95	100	61	83	87	48	64	71
7	5,490	5,870	5,860	94	100	111	81	94	105	72	87	102	52	70	88
8	5,490	5,870	5,860	101	106	117	89	107	118	83	105	117	62	81	108
9	4,900	5,420	5,790	107	104	103	97	106	106	77	94	107	53	68	83
10	5,490	5,870	5,860	96	104	102	85	95	99	69	85	87	52	74	87
11	6,100	5,970	6,190	89	104	109	77	92	102	58	80	89	36	58	79
12	5,390	5,670	6,070	98	102	108	93	106	112	73	86	105	59	79	98
13	5,490	5,870	5,860	93	96	97	76	79	95						
13	4,900	5,420	5,790							65	73	82	43	55	68
14	5,440	5,610	5,640	90	92	94	79	87	98	59	75	86	43	59	72
15	4,900	5,420	5,790	98	95	94	84	92	93	65	75	83	41	54	69
16	6,100	5,970	6,190	88	98	102	80	98	108	71	90	106	58	83	108
17	5,080	5,360	5,650	104	107	111	92	102	106	80	93	105	57	78	94
18	5,410	5,220	5,750	103	107	115	97	112	112	73	98	108	55	76	88
19	5,080	5,360	5,650	99	103	103	85	98	102	57	72	81	41	67	77
20	5,410	5,220	5,750	93	104	101	82	95	104	60	77	89	42	61	72
21	5,410	5,220	5,750	85	95	88	67	79	75	42	58	57	25	38	42
22	5,080	5,360	5,650	93	96	99	78	93	97	56	69	80	36	47	63
23	5,570	5,640	5,940	87	92	93	67	79	86	49	67	78	31	48	62
24	5,570	5,640	5,940	85	90	87	72	82	90	59	74	91	36	57	75
25	5,250	5,270	5,530	91	98	95	76	84	83	58	71	75	37	50	55
26	5,080	5,360	5,650	92	97	97	77	86	91	57	73	81	37	54	65
27	5,670	5,740	5,770	90	96	97	72	77	82	49	59	65	38	50	55
28	6,100	5,970	6,190	86	95	95	72	85	88	54	69	77	36	54	67
29	5,440	5,610	5,640	89	91	109	80	87	101	58	68	85	38	50	67
31	5,540	5,500	5,490	98	105	111	79	93	106	65	85	109	48	74	97
32	5,540	5,500	5,490	91	97	102	69	78	84	47	63	79	28	42	58
33	5,540	5,500	5,490	94	97	102	68	75	75	44	56	64	31	42	50
34	5,670	5,740	5,770	91	100	99	74	84	86	49	61	67	34	46	51
Average				93	99	102	80	92	98	63	79	90	46	64	78
1 : 2 MORTAR															
1	8,680	8,940	9,740	84	91	95	80	91	99	70	87	101	60	79	86
3	7,490	8,120	9,420	100	104	104	99	114	109	92	113	110	85	102	98
14	8,680	8,940	9,740	94	101	101	81	93	97	60	76	84	44	61	71
29	8,680	8,940	9,740	92	98	99	78	88	92	58	71	79	34	48	61
Average				92	98	100	84	96	99	70	87	94	56	72	79
1 : 3.5 MORTAR															
1	4,140	4,050	4,270	87	103	105	76	98	103	66	90	110	57	80	103
3	4,140	4,050	4,270	92	101	105	87	108	116	72	103	125	66	99	119
14	4,140	4,050	4,270	90	99	96	78	91	99	53	72	87	37	57	80
29	3,890	4,090	4,160	93	96	98	75	86	90	55	68	80	36	49	65
Average				90	100	101	79	96	102	62	83	100	49	71	92

[1] Moist air storage at 73° F.
[2] Each value is an average of three 2-inch cubes.
[3] Reported as a percentage of the strength of control mortar.

the amount of water required for uniform consistency of cement-fly ash mortars was not constant. The effect of each fly ash on the water requirement of a 1:2.75 mortar is shown in figure 13, where the samples are arranged from left to right in increasing order of their carbon content. As stated before, the water requirement ratio is the amount of water used to prepare a mortar containing fly ash expressed as a percentage of the amount of water used to prepare the corresponding control mortar. The ratios plotted are average values for the 35-percent replacement mortars prepared with all three cements.

Some fly ashes which contained very little carbon improved the workability of the mortar to such an extent that less water was required for the mortar containing fly ash than for the control mortar. The quantities of water required for mortar of uniform consistency generally increased with the carbon content of the fly ash, although the departures from this trend indicate that differences in physical characteristics also affect the water requirement as would be expected.

The relation between the pozzolanic strength index and the water requirement ratio is shown in figure 14, and appears to be somewhat better than that obtained with either carbon content (fig. 12) or fineness as determined by the No. 325 sieve (fig. 11). This would appear to be logical, since both the carbon content and fineness of fly ash tend to affect the amount of water needed for a uniform consistency of mortar.

Relation of Chemical Constitution to Pozzolanic Strength Index

The pozzolanic strength indexes were plotted against the total amounts of each of the inorganic constituents previously given in table 1. These include alumina, iron oxide, calcium oxide, magnesium oxide, sulfur trioxide, alkalies as equivalent sodium oxide, and silica. No indication of a definite relation was found in any of these plots. Even though no relations were found, plots of pozzolanic strength index against total alkalies as Na_2O and silica are shown in figure 15 because of the general interest in these constituents.

Considering the general relation found between the alkali content of cement and mortar strength ratios at 28 days (table 9), it might be thought that the alkalies in the fly ash would be a significant factor. Figure 15 shows that those fly ashes with the highest alkali content had high pozzolanic strength indexes, but for most of the materials the strength index appears to be independent of the alkali content. The silica in fly ash is of interest, as it might be expected from the definition of a pozzolanic material that this constituent

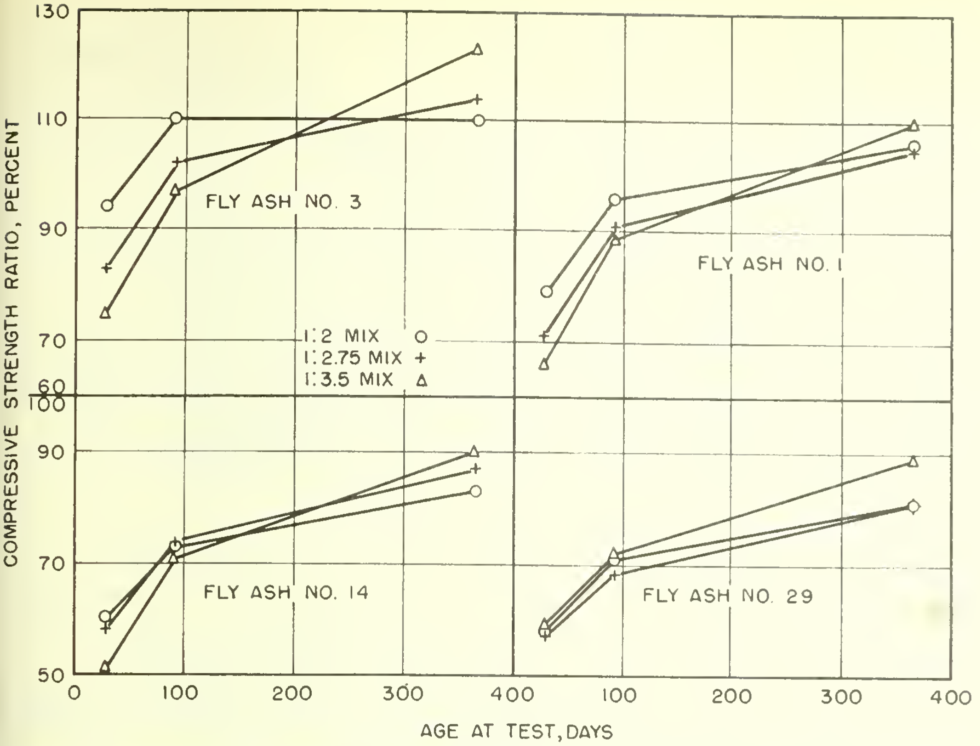

Figure 6.—Effect of mix proportions on strength of cement-fly ash mortar with 4 fly ashes used as replacement for 35 percent of the cement.

would have some effect on the strength of mortar. It is possible that silica may influence the strength of mortar to some extent, but the scattered pattern obtained in figure 15 shows that there is no direct relation that applies generally to all fly ashes.

Use of Fly Ash To Prevent Expansion by Alkali-Aggregate Reaction

A frequent cause of failure in concrete is the expansion and cracking that may result from the attack of certain reactive siliceous constituents of the aggregate by the alkalies in the cement. It has been found that the destructive effects of this reaction can be prevented by the addition to the concrete of a very finely divided siliceous material which is itself reactive with the alkalies in cement.

The effectiveness of 17 fly ashes in preventing or reducing the expansion resulting from the alkali-aggregate reaction was determined by measuring the change in length of 1- by 1- by 11¼-inch mortar bar specimens stored in moist air at 100° F. Control specimens were prepared from a basic reactive mortar consisting of a 1:2 mix by weight using graded Ottawa sand containing 2 percent of reactive opal passing the No. 8 and retained on the No. 50 sieve. Each fly ash was used to replace 10, 20, 35, or 50 percent of the cement on a solid volume basis. In all other respects the procedure outlined in the test for Potential Alkali Reactivity of Cement-Aggregate Combinations (ASTM Designation C 227–51 T) was followed. All cement-fly ash combinations with opal added were prepared with both cements A and B, which contained 0.90 and 0.13 percent equivalent Na_2O, respectively. In addition, specimens without opal were prepared with both cements and various percentages of each fly ash.

All specimens prepared either without opal or with the low-alkali cement showed expansions of less than 0.05 percent at an age of one year and are not reported here. The expansion of mortars containing opal and prepared with cement A (0.90 percent alkali) is shown in table 11 for ages of one month and one year. All specimens prepared with the same fly ash were made on the same day with a set of control specimens and were stored and tested together. Although the different sets of control specimens failed to develop the same amount of expansion, each set may be used to judge the effect of the single fly ash to which it applies.

It is evident that any of the fly ashes tested will prevent the expansion caused by the alkali-aggregate reaction if sufficient quantity is used. At an age of one year, for example, replacement of 35 or 50 percent of the cement by any of the fly ashes limited the change in volume to a maximum of 0.04 percent which is comparable with that shown by nonreactive mortars. All specimens have been measured to an age of 2 years, but no significant increase in volume has occurred in those having a replacement of 35 or 50 percent fly ash. Although a replacement of 20 percent of the cement by fly ash reduced the expansion at one year, it did not reduce it to a safe amount in all cases. The reduction effected by a 20-percent replacement varied considerably for different fly ashes, indicating that they are not equally effective in preventing the expansion resulting from the alkali-aggregate reaction. This difference in effectiveness is illustrated in figure 16, p. 136.

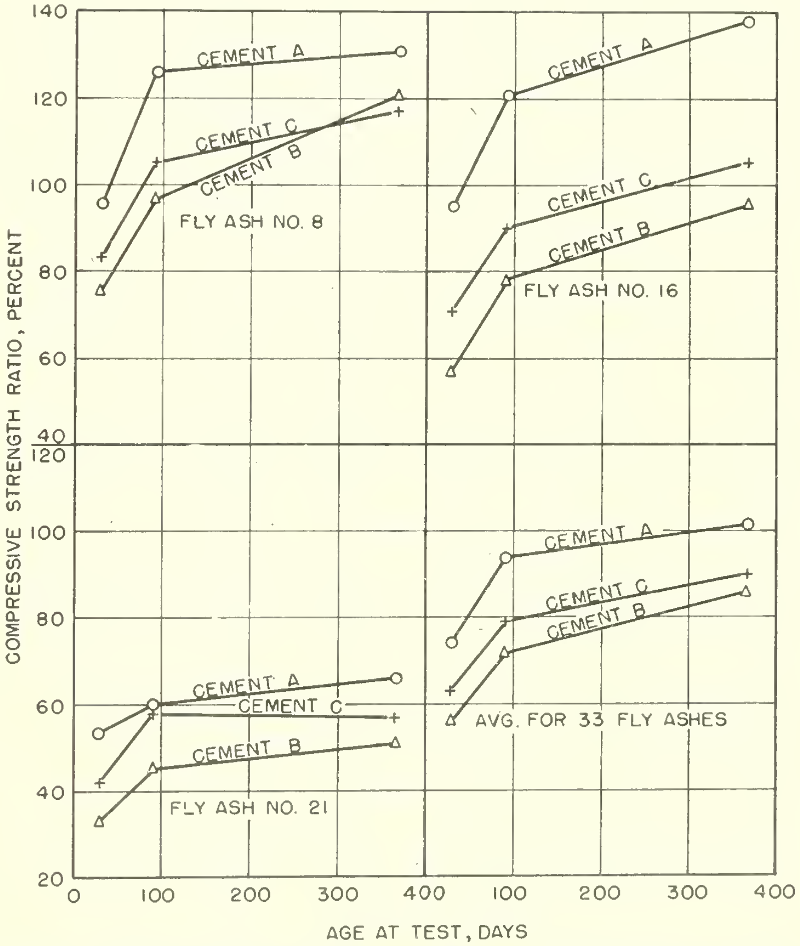

Figure 7.—Effect of cement on strength of cement-fly ash mortar (1:2.75 mix, 35-percent replacement of cement with fly ash).

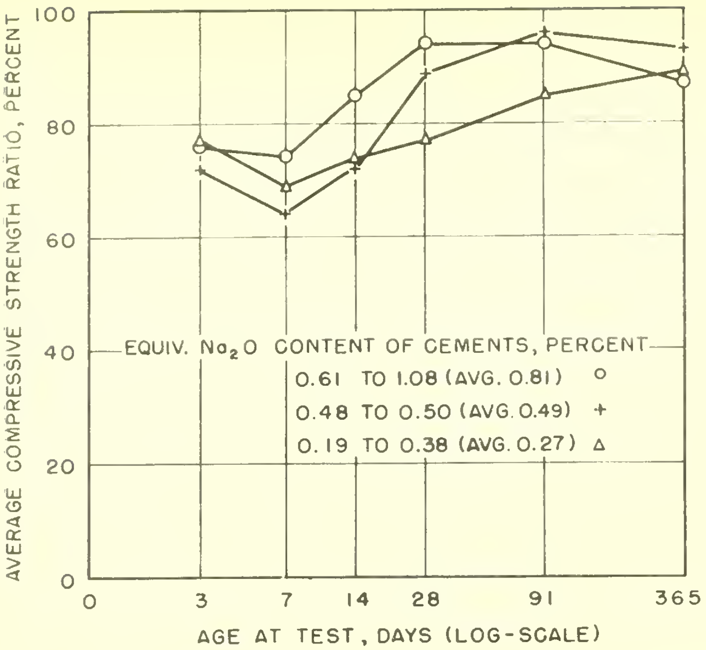

Figure 8.—Effect of alkali content of cement on strength of cement-fly ash mortar (1:2 mix, 50-percent replacement of cement with fly ash).

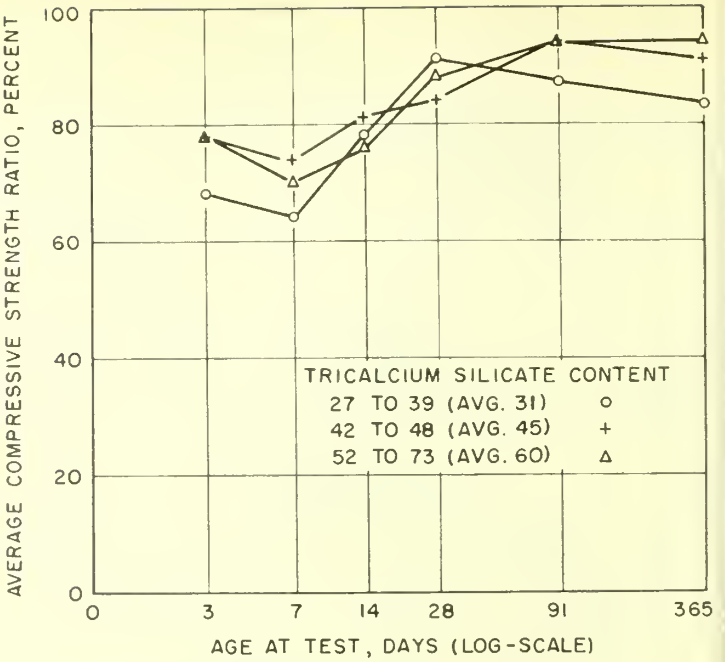

Figure 9.—Effect of tricalcium silicate content of cement on strength of cement-fly ash mortar (1:2 mix, 50-percent replacement of cement with fly ash).

For the particular reactive mortar used in these tests, a 10-percent replacement of cement with fly ash did not produce a substantial reduction in expansion, and in some cases, the change in volume was increased. At an age of one year, specimens prepared with 7 of the 17 fly ashes in the 10-percent replacement group showed more expansion than the control specimens.

It should be emphasized that the mortar tests upon which this discussion is based may not reflect the actual behavior of materials when used in concrete. At the present time, tests of mortar bars cannot be relied upon as substitutes for tests of concrete specimens to determine the amount of a particular fly ash which must be used to prevent expansion in a reactive concrete.

Special Chemical Studies

Since strength tests of mortar are time consuming, there has been considerable interest in the development of more rapid tests to measure the activity of pozzolans. Most of these tests have attempted to measure in some way special chemical properties of the pozzolans alone or the chemical changes occurring in mixtures of lime and pozzolans. In their paper on determining pozzolanic activity, Moran and Gilliland (*8*) give a summary of most of these tests. They point out in summation that "a single short-time test will not evaluate pozzolanic activity, particularly when any one of several properties may be desired in a given material. It appears that each material requires rather exhaustive testing, after which it may be possible to empirically relate a quick test for control purposes." The special chemical tests made in this investigation were conducted with the latter objective in mind. They were undertaken to supply a background of information for further work so that ultimately it might be possible to establish interrelations between various properties and test results which would make possible more rapid methods of test.

The special tests conducted on the fly ash itself involved chiefly the determination of constituents soluble in water or sodium hydroxide, and the changes in solubility resulting from reactions of lime and fly ash in the presence of water.

Table 12 shows the amount and calculated composition of the water soluble material in each of the fly ashes. These data were obtained by shaking a 5-gram sample of fly ash in 100 ml. of distilled water for one hour, allowing the mixture to stand 24 hours and filtering the supernatant liquid without further dilution or washing. The pH of this extract was determined and is also recorded in table 12.

Chemical analyses were conducted on aliquots for the amounts of calcium, sodium, potassium, and sulfate. The total amount of dissolved material was also determined by evaporating the water from an aliquot and drying the residue at 110° C. The composition of the dissolved salts was calculated by assuming that all of the sulfate was combined as gypsum ($CaSO_4 \cdot 2H_2O$). The balance of the calcium was then calculated as calcium hydroxide ($Ca(OH)_2$). The sodium and potassium were assumed to be present as the hydroxides (NaOH or KOH). Generally there is good agreement between the total amount of soluble material and the total amount of calculated salts. This is an indication of the validity of the assumptions made in calculating the composition.

The pH values of the extracts depend on the proportions of water and sample used in the test as well as the composition of the dissolved material, and there is no direct relation

Table 9.—Effect of cement on the compressive strength ratio of cement-fly ash mortar

Cement identification	Properties of cements				Compressive strength ratio of 1:2 mortars with 50-percent replacement of cement with fly ash No. 3 after—					
	Alkali content, equivalent Na_2O	Tricalcium silicate content	Fineness by turbidimeter	Compressive strength of 1:2 control mortar at 28 days	3 days	7 days	14 days	28 days	91 days	1 year
	Pct.	*Pct.*	*cm²/gm.*	*P. s. i.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
M	1.08	27	2,100	5,025	68	62	81	96	94	83
L	.82	48	1,860	6,365	83	87	95	94	97	89
K	.74	28	2,000	5,210	61	66	80	89	80	77
J	.61	52	1,800	6,600	90	81	85	98	104	100
Average	.81	39	1,940	5,800	76	74	85	94	94	87
I	.50	55	1,870	6,610	72	67	74	95	103	98
H	.48	73	2,440	8,540	69	62	71	85	99	92
G	.48	39	1,760	6,415	76	64	72	88	87	90
Average	.49	56	2,025	7,190	72	64	72	89	96	93
F	.38	42	1,860	7,950	81	68	71	74	92	85
E	.24	46	2,040	7,135	69	67	76	85	93	99
D	.19	61	1,780	8,125	81	72	76	72	70	84
Average	.27	50	1,890	7,740	77	69	74	77	85	89

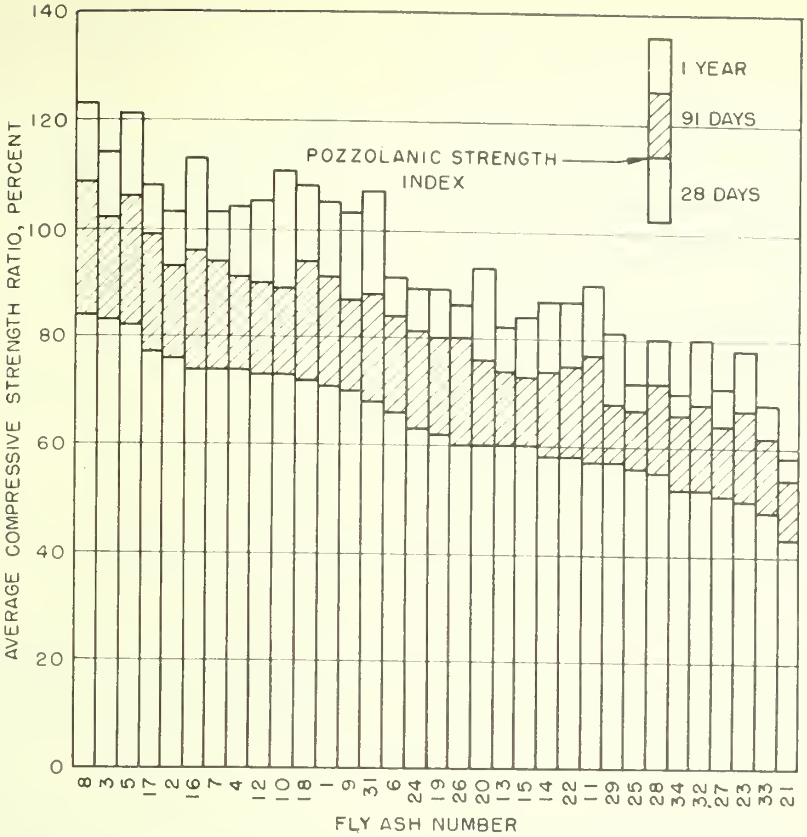

Figure 10.—Development of compressive strength in cement-fly ash mortar (1:2.75 mix, 35-percent replacement of cement with fly ash).

between the quantitative amount of any particular constituent and the pH. Generally the values obtained are those to be expected from a consideration of the calculated amount of calcium hydroxide present. In table 12, the pH value is relatively low for those extracts having no calcium hydroxide by calculation and high for those extracts in which the calculated calcium hydroxide is large. For all samples that have a calculated lime content greater than 2.0 percent, the pH of the extract was 12.0 or more.

The data for the soluble salts in table 12 show that essentially all of the extracted water soluble material is either calcium sulfate or calcium hydroxide, the relative amounts of each varying greatly for different fly ashes. These data, however, do not indicate the state in which the soluble materials are present in the original fly ash. The calcium sulfate may be present in any of its three forms—anhydrous, hemihydrate, or dihydrate—depending upon the conditions surrounding the fly ash during storage or handling. Likewise the calcium hydroxide may be present as uncombined lime or may have formed from the hydrolysis of lime-silica complexes after the addition of water.

To verify the presence of uncombined lime in the fly ash prior to wetting with water, determinations were made by the alcohol-glycerol method, such as is used in the determination of free lime in portland cement (ASTM Method C 114–53). The results of these tests are given in table 13. This table also includes the total amount of calcium that is present in the sample and the amount that is water soluble. The part of the water soluble calcium that is calcium hydroxide and the part that is calcium sulfate are also given. All these results are expressed in terms of an equivalent amount of calcium oxide to provide a common basis of comparison. It can be seen that for most of the samples the determined free lime is approximately the same as the calculated water soluble calcium hydroxide, thus indicating that the addition of water to fly ash does not result in the formation of calcium hydroxide by the hydrolysis of lime complexes. It will also be noted that a considerable portion of the calcium is not soluble in water, and therefore is probably present as a calcium glass.

The data in table 14 were obtained by making the test for reactivity of pozzolans with sodium hydroxide solution as described in Appendix A of the report by Moran and Gilliland on determining pozzolanic activity (*8*). This test is an adaptation of a procedure now designated as the Tentative Method of Test for Potential Reactivity of Aggregates (Chemical Method), ASTM Designation C 289–54 T. It involves treatment of the sample with 1 N. solution of sodium hydroxide in a sealed container at 80° C. for a 24-hour period. At the end of this period, the solution is filtered and the resulting filtrate examined. In the tests described by Moran and Gilliland, only the reduction in alkalinity (the amount of sodium hydroxide consumed by the reactions taking place) and the amount of silica dissolved were determined. In this investigation the amounts of alumina and sulfate dissolved were also determined.

This test was originally proposed for use on calcined shales. For this type of material it showed some promise in evaluating the ability of the pozzolan to prevent expansion resulting from the alkali-aggregate reaction. Consequently, consideration has been given to its application to fly ash. It was found however that the presence of calcium sulfate in a fly ash affects the results obtained by this test. When calcium sulfate dissolves, the calcium ion precipitates much of the silica which might normally be soluble and remain in solution if no calcium were present. The presence of dissolved calcium sulfate might also affect the reduction in alkalinity by causing a precipita-

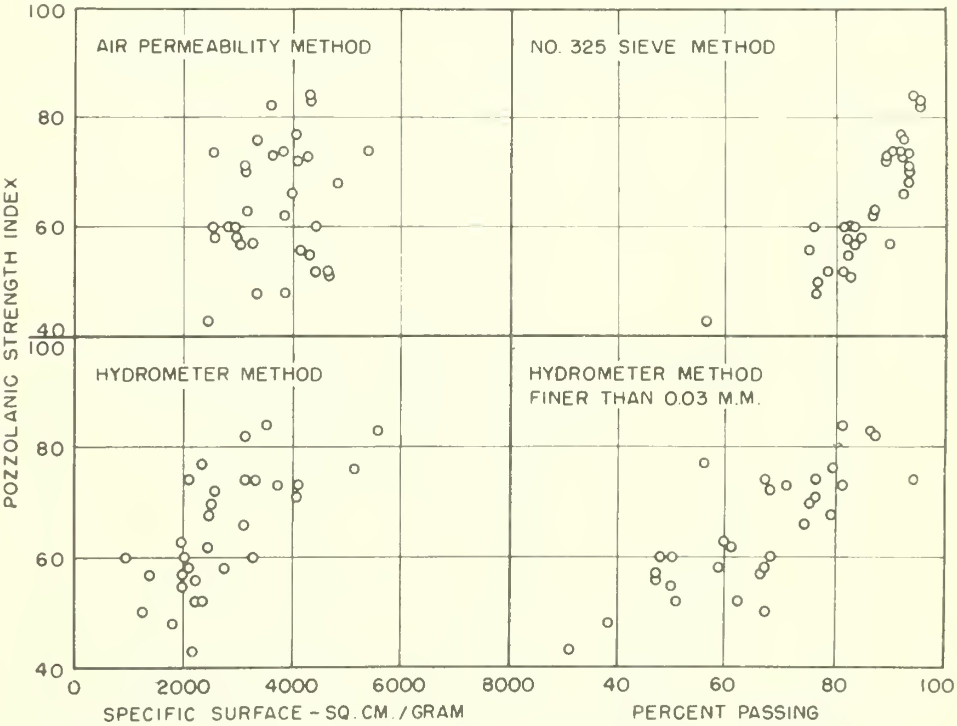

Figure 11.—Comparison of fineness of fly ash and pozzolanic strength index.

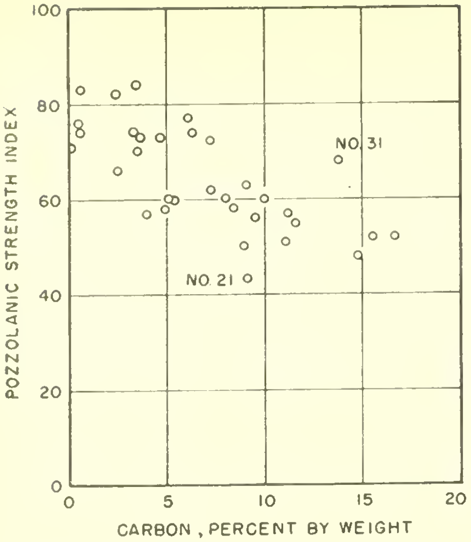

Figure 12.—Comparison of carbon content of fly ash and pozzolanic strength index.

tion of calcium hydroxide from solution. The data in table 13 illustrate these effects.

In most cases where the calcium sulfate dissolved is relatively high (as indicated by the amount of SO_3), the amount of silica dissolved is low and the reduction in alkalinity is relatively high. It appears that the relation between soluble silica and reduction in alkalinity is controlled by the amount of calcium sulfate present as well as by the characteristics of the siliceous material. Values for reduction in alkalinity and amount of silica dissolved which have been found suitable for acceptance requirements of other types of pozzolans may not be applicable to fly ashes.

A plot of the pozzolanic strength index against the reduction in alkalinity failed to show any correlation. An apparent relation between the strength developed and the silica dissolved is shown in figure 17, but this relation is the inverse of what might normally be expected, that is, that larger amounts of dissolved silica would result in greater pozzolanic activity. As previously stated, the presence of calcium sulfate controls to a considerable extent the amount of silica remaining in solution. For this reason no particular significance can be attached to this apparent relation.

The amount of Al_2O_3 dissolved varied from 0.03 to 0.41 percent, the median being 0.1 percent. No significance could be attached to the variations in these values.

The data given in tables 15–18 were obtained by analysis of a slurry of fly ash, lime, and water that had been stored in an oven at 100° F. for periods of 7, 28, and 91 days. The procedure used for the preparation and storage of the slurries was essentially the same as that given for the determination of "available alkalies" in the Tentative Methods of Sampling and Testing Fly Ash for Use as an Admixture in Portland Cement Concrete, ASTM Designation C 311–54 T. This test has been proposed as a relative measure of the amounts of alkalies from the fly ash that are made available for reaction with any reactive aggregate that may be present in the concrete. For this study tests were made on slurries at ages of 7 and 91 days as well as the 28 days called for in the ASTM method. Some variation in technique from the ASTM method was found necessary to insure better control of test conditions; the details of the test procedure used for this investigation are given in Appendix A (procedure 2).

Table 15 shows the individual amounts of sodium and potassium oxides that were extracted with water after each test period and the total of these two expressed as an equivalent amount of sodium oxide. Each result is expressed as a percentage of the total amount of the constituent present in each fly ash. The total amounts of alkalies present in each sample and the alkalies soluble at 28 day expressed as percentages of the original weigh of fly ash are also given. These results in variably show considerable increase in th solubility of alkali between 7 and 28 days but the results for the 91-day period diffe with the samples tested. Some show littl or no further increase, but others show marked increase in the alkali dissolved This increase was as much as 28 percent o the equivalent (total) alkali in the case o sample 23. The increase was greater tha 10 percent in 18 of the 34 samples. It i evident that storage of lime-fly ash slurry fo 28 days does not yield a result for solubl alkalies that can be related to the ultimat solubility of the alkalies in lime solution.

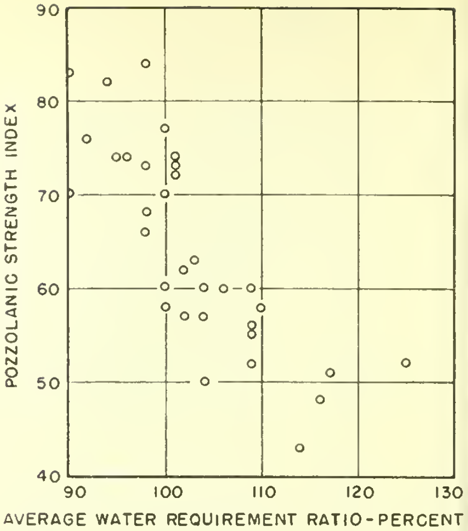

Figure 14.—Comparison of water requirement ratios of fly ash and pozzolani strength index.

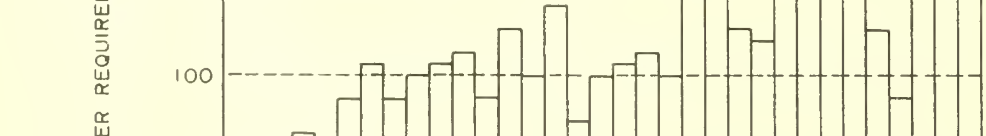

Figure 13.—Water requirement ratios in cement-fly ash mortar for 33 fly ashes (1:2.75 mix, 35-percent replacement of cement with fly ash).

In order to obtain additional informatio of the chemical changes taking place in inti mate mixtures of fly ash and lime in th presence of water, the residues after the ex traction of the alkalies were digested in dilut hydrochloric acid (1:10). The acid solutio was filtered off and the residue from thi treatment digested in sodium hydroxide (percent). This was then filtered. Each o the filtrates thus obtained was examined fo the amounts of silica, alumina, and iron oxid present. The details of the procedure use are described in Appendix A (procedure 2 These results are shown in tables 16–1 respectively.

Because of the exploratory nature of thes tests, relatively large experimental errors a likely but the results are of interest becaus they show that the solubility of the alumin (table 17) was increased to a much great extent during the period covered by the tes than was the solubility of the silica (table 16 Table 16 shows that for some samples the is a general trend toward larger amounts o acid-soluble silica being present for the long period of storage, but the results are errati In all cases only a small fraction of the tot amount of silica present was soluble in aci The change with age in the acid solubility o the alumina shown in table 17 is considerabl The acid soluble alumina at 7 days is in t range of 2 to 4 percent for most samples, b

Table 10.—Effect of carbon in fly ash on the compressive strength of cement-fly ash mortar

Fly ash number	Method of replacing cement with fly ash [1]	Carbon content of fly ash	Water requirement ratio, based on control mortar	Compressive strength ratio, based on 1:2.75 control mortar at—	
				28 days	91 days
		Percent	*Percent*	*Percent*	*Percent*
1	1	0.2	84	89	108
	2	.0	82	87	108
	3	.0	83	86	104
7	1	3.5	96	80	99
	2	.0	86	74	85
	3	.0	86	76	90
14	1	5.0	100	66	76
	2	.0	88	71	84
	3	.0	88	68	83
29	1	11.2	96	65	81
	2	.0	86	85	106
	3	.0	86	68	86
32	1	14.3	104	57	69
	2	.0	90	71	90
	3	.0	90	61	75

[1] In the three methods used, 35 percent of the solid volume of cement in the control mortar was replaced as follows: *Method 1*, by an equal volume of fly ash as received; *method 2*, by an equal volume of fly ash after ignition at 550° C.; and *method 3*, by an amount of ignited fly ash equal in volume to that of the inorganic portion of the fly ash used in method 1.

at 91 days this range is 7 to 10 percent. Generally, the increase in the solubility of the alumina between 7 and 91 days represented from 20 to 30 percent of the total alumina in the sample. The solubility of the iron oxide shown in table 18 indicates somewhat the same trend as that of the alumina but to a lesser degree.

Because of the considerable increase with time in the solubility of alumina in the presence of lime and water, the relation of this acid-soluble alumina to the pozzolanic strength index is of interest. Figure 18 shows the amount of acid-soluble alumina after 91 days plotted against the pozzolanic strength index. There is a general trend toward higher strength ratios for larger amounts of acid-soluble alumina, but it is also apparent that samples having widely different amounts of acid-soluble alumina have essentially the same strength index. Although the reactivity of the alumina may be a factor in the development of strength, other factors must exert a greater influence.

The amounts of alkali-soluble material shown in tables 16–18 have little significance. This alkali digestion was made primarily to determine whether significant changes occurred in the amount of amorphous silica present in the fly ash during reaction with lime. The test procedure was based on the assumption that only this form of silica would be insoluble in the acid treatment and soluble in the alkali solution. However, the glassy constituents of the fly ash were evidently decomposed to some extent by the conditions of the test and no definite trend could be established.

Lime-Fly Ash Tests for Measuring Pozzolanic Activity

The development of strength as a result of pozzolanic action requires that lime be available for reaction with the active constituent of a pozzolanic material. Some of the earliest methods of evaluating pozzolanic activity included determination of the effect of the reaction taking place in pozzolan-lime mixtures. Data obtained using three such procedures are shown in table 19.

In one method, lime-fly ash mortar was cured at temperatures of 100° and 130° F. and the compressive strength determined at an age of 7 days. Full details of this test as performed in this investigation are given in procedure 3 of Appendix A. Except for fly ash samples 1, 2, and 3, higher strengths were obtained by curing the specimens at 130° F. than at 100° F. The data show that the higher curing temperature is generally of marked benefit to the less active fly ashes. A comparison between the pozzolanic strength index and the compressive strength of lime-fly ash mortar cured at 100° F. failed to show any correlation. A reasonable relation between the index and the strength of the lime-fly ash mortar cured at 130° F. is shown in figure 19. However, the six fly ashes identified by number in figure 19 do not follow the general trend.

Alexander and Wardlaw (*9*) have indicated that lime mortar tests of pozzolanic materials conducted for a fixed period at constant temperature may not provide a proper measure of the reactivity of such materials. It is possible that fly ashes 4, 7, 8, 12, and 17 would have shown greater reactivity with lime if the test had been made under other conditions of storage such as higher temperature or longer curing.

The second method involved the determination of the time of setting of a lime-fly ash mixture. Moran and Gilliland (*8*) described this method as developed by Feret and used for evaluating pozzolanic materials other than

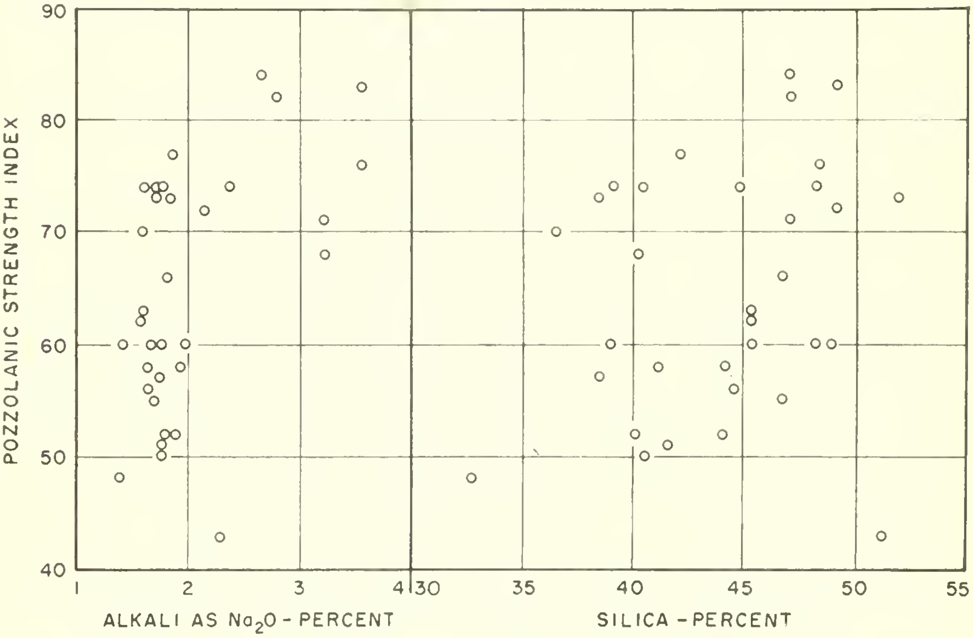

Figure 15.—Comparison of alkali or silica content of fly ash and pozzolanic strength index.

Table 11.—Effect of fly ash on the volume change of an alkali-reactive mortar containing opal

Fly ash number	Expansion of mortar bars with 0-50 percent replacement of cement with fly ash and stored at 100° F.[1]									
	Age of 1 month					Age of 1 year				
	None	10	20	35	50	None	10	20	35	50
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
4	0.10	0.03	0.02	0.00	0.01	0.62	0.54	0.19	0.01	0.00
5	.12	.07	.02	.00	.00	.53	.39	.09	.01	.01
6	.07	.04	.02	.02	.01	.38	.30	.17	.02	.01
7	.11	.12	.02	.01	.01	.46	.46	.18	.01	.01
8	.06	.12	.02	.01	.02	.41	.40	.11	.02	.02
9	.14	.12	.00	.01	.00	.52	.43	.09	.00	.00
10	.02	.03	.01	.00	.00	.44	.45	.15	.01	.00
11	.09	.08	.02	.02	.02	.39	.40	.12	.02	.01
12	.02	.04	.02	.00	.01	.56	.35	.04	.01	.02
13	.05	.10	.01	.00	.02	.63	.48	.20	.00	.02
15	.08	.05	.00	.00	.00	.50	.47	.17	.02	.01
16	.02	.07	.02	.01	.02	.54	.55	.27	.02	.03
19	.10	.06	.00	.00	.01	.43	.45	.13	.01	.02
25	.04	.12	.02	.02	.02	.40	.54	.25	.03	.02
27	.05	.10	.04	.02	.01	.44	.46	.39	.02	.01
28	.06	.10	.02	.00	.02	.38	.37	.08	.01	.01
34	.09	.10	.02	.01	.02	.45	.49	.30	.04	.01
Average	.07	.08	.02	.01	.01	.48	.44	.17	.02	.01

[1] Specimens prepared with cement A (equivalent Na_2O=0.90 percent). Each value is an average for two specimens.

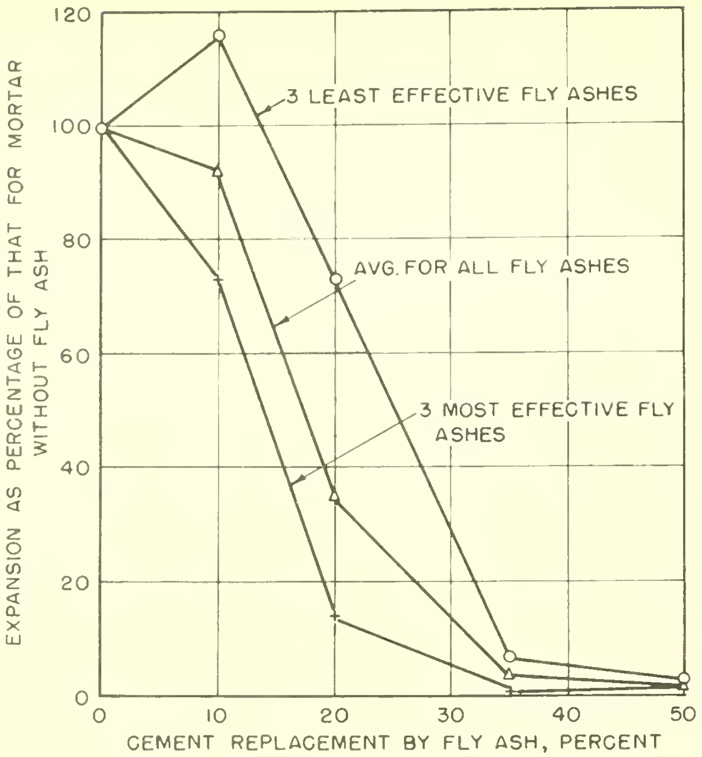

Figure 16.—Effect of fly ash on expansion of alkali-reactive mortar at age of 1 year.

fly ash. The details of the test are given in procedure 4 of Appendix A. When the initial and final setting times shown in table 19 are compared with the pozzolanic strength indexes, a general trend towards an inverse relation is found, but the relation is not sufficiently definite to make the test of much value for determining the quality of fly ash.

Moran and Gilliland (*8*) also described a lime absorption test to measure the activity of a pozzolan. This test provides for the measurement of the amount of lime absorbed from a saturated lime solution over periods of time. The absorption of lime was measured by the decrease in the amount of acid required to neutralize an aliquot of the solution. The difference between the amounts absorbed at 7 and 28 days was then taken as a measure of the pozzolanic activity of the material. The details of the test as performed in this investigation are given in procedure 5 of Appendix A. Moran and Gilliland showed good correlation between the results of this test and the strength of portland cement-pozzolan mortars when the pozzolans were volcanic ash or pumice. The results obtained with this test on the fly ashes used in this investigation are shown in table 19.

A comparison of these results with the pozzolanic strength indexes failed to show any correlation. For a number of samples, negative values were obtained. This is not surprising as it was shown in the tests of lime-fly ash slurries that the alkalies in fly ash are slowly soluble in the presence of lime. Although the use of the 7-day result as the starting point would eliminate the effect of rapidly soluble constituents, the continued solubility of constituents in the fly ash after that time affects the equilibrium conditions with respect to lime and alkalies in solution. This test is considered not applicable to fly ashes.

Table 12.—Alkalinity and composition of water extracts of fly ash

Fly ash number	pH of extract	Calculated composition of dissolved material [1]				Total dissolved material [1]		
		$CaSO_4.2H_2O$	$Ca(OH)_2$	NaOH	KOH	Calculated	Determined (dried at 110° C.)	Difference (determined less calculated)
		Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
1	11.8	3.66	1.22	0.23	0.06	5.17	5.16	−0.01
2	11.7	3.02	.65	.34	.07	4.08	4.08	.00
3	11.7	4.41	.18	.45	.11	5.15	5.19	.04
4	11.8	2.02	1.43	.03	.06	3.54	3.49	−.05
5	11.8	2.87	1.00	.18	.06	4.11	3.93	−.18
6	11.8	3.12	1.44	.06	.06	4.68	4.55	−.13
7	11.0	2.18	.13	.05	.10	2.46	2.54	.08
8	11.7	1.75	.18	.08	.06	2.07	2.14	.07
9	12.0	1.59	2.02	.05	.02	3.68	4.48	.80
10	12.3	2.97	3.80	.04	.05	6.86	6.65	−.21
11	10.6	.95	.15	.03	.02	1.15	1.30	.15
12	11.3	.79	.33	.03	.02	1.17	1.30	.13
13	10.1	.47	.04	.01	.02	.54	.73	.19
14	12.0	1.13	1.25	.08	.02	2.48	2.94	.46
15	11.9	1.29	1.66	.04	.02	3.01	3.15	.14
16	12.3	2.68	4.48	.04	.02	7.22	7.15	−.07
17	11.4	1.84	.42	.05	.02	2.33	2.28	−.05
18	11.3	.97	.30	.04	.05	1.36	1.58	.22
19	9.9	.64	.03	.03	.04	.74	.77	.03
20	12.1	.40	2.59	.01	.02	3.02	3.92	.90
21	11.3	.47	.46	.03	.04	1.00	1.22	.22
22	12.3	1.46	3.74	.01	.02	5.23	5.31	.08
23	12.0	2.42	2.92	.04	.02	5.40	5.34	−.06
24	9.0	.65	.00	.04	.00	.69	.70	.01
25	10.4	1.54	.17	.01	.02	1.74	1.78	.04
26	8.5	.73	.00	.04	.05	.82	.78	−.04
27	12.0	3.19	2.30	.06	.06	5.61	5.45	−.16
28	9.3	.64	.00	.03	.02	.69	.75	.06
29	11.1	1.04	.41	.03	.01	1.49	1.62	.13
30	11.9	.89	.45	.08	.02	1.44	1.67	.23
31	11.0	4.35	.07	.34	.22	4.98	4.80	−.18
32	10.8	.43	.12	.03	.02	.60	.68	.08
33	10.0	2.81	.17	.03	.06	3.07	3.12	.05
34	11.2	3.03	.65	.03	.00	3.71	3.84	.13

[1] Expressed as a percentage of the original weight of fly ash.

Table 13.—The distribution of the calcium in fly ash

Fly ash number	Calcium [1] as CaO from—				
	Total sample	Total water-soluble	Water-soluble $CaSO_4$	Water-soluble $Ca(OH)_2$	Free lime [2]
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
1	7.0	2.1	1.2	0.9	0.9
2	5.2	1.5	1.0	.5	.5
3	5.5	1.6	1.4	.2	.0
4	5.2	1.7	.7	1.0	.9
5	5.2	1.7	.9	.8	.6
6	6.4	2.1	1.0	1.1	.8
7	1.7	.8	.7	.1	.2
8	4.5	.7	.6	.1	.5
9	7.8	2.0	.5	1.5	1.6
10	8.3	3.9	1.0	2.9	2.3
11	1.8	.4	.3	.1	.0
12	2.3	.5	.3	.2	.1
13	1.6	.2	.2	.0	.0
14	6.0	1.3	.4	.9	1.4
15	6.4	1.7	.4	1.3	1.0
16	11.6	4.3	.9	3.4	2.6
17	4.3	.9	.6	.3	.2
18	3.6	.5	.3	.2	.2
19	1.4	.2	.2	.0	.0
20	5.2	2.1	.1	2.0	1.5
21	1.6	.5	.1	.4	.3
22	7.6	3.3	.5	2.8	2.3
23	8.5	3.0	.8	2.2	1.6
24	1.5	.2	.2	.0	.0
25	6.2	.6	.5	.1	.0
26	1.2	.2	.2	.0	.0
27	6.8	2.8	1.0	1.8	1.0
28	1.1	.2	.2	.0	.0
29	3.2	.6	.3	.3	.1
30	12.0	.6	.3	.3	.9
31	3.9	1.5	1.4	.1	.0
32	1.7	.2	.1	.1	.0
33	1.8	1.0	.9	.1	.0
34	3.5	1.5	1.0	.5	.0

[1] Expressed as a percentage of the original weight of fly ash.
[2] Determined by ASTM Method C 114-53.

Test Methods and Specification Requirements for Fly Ash

The determinations made on fly ash in this study generally follow procedures which have been used previously for testing either fly ash or other types of pozzolans. The significance of each of these determinations for measuring the quality of fly ash has been assessed by comparison of the effects of fly ash on the strength of mortar. In the following discussion the test methods and specification requirements which are usually applied to fly ash are evaluated in view of the data obtained in this investigation.

The results of these tests indicate that the effect of fly ash on the strength of mortar is related to the carbon content of the fly ash, the fineness as determined by use of the No. 325 sieve or the 0.03 mm. size in the hydrometer analysis, and the water requirement ratio. The use of the No. 325 sieve is preferred over the hydrometer method for determination of fineness because of the relative simplicity of the method. Limiting values for these properties which were found for fly ashes having various degrees of pozzolanic activity are shown in table 20. This information can be used for making a quick evaluation of a fly ash in terms of its carbon content, loss on ignition, fineness, and water requirement ratio. The data in this table may also serve as a basis for preparing a specification for fly ash for use in concrete.

Although the data in table 20 indicate that fly ashes of a selected quality as measured by the strength index will generally meet certain limits for carbon content, loss on ignition, fineness, and water requirement ratio, it should not be inferred that limits on these properties alone will always insure a particular level of pozzolanic activity. Since the relations existing between pozzolanic activity and these properties are not exact, some fly ashes may not be as active as an appraisal based on these properties would indicate. Fly ash No. 6, for example, with a loss on ignition of 4.1 percent, 92.1 percent passing the No. 325 sieve, and a water requirement ratio of 98 percent, would be expected on the basis of table 20 to have a higher strength index than 66 as shown in figure 10. This shows that limits on these properties of fly ash do not eliminate the need for a more positive test for pozzolanic activity.

Table 14.—Reaction of fly ash with sodium hydroxide solution [1]

Fly ash number	Reduction in alkalinity of solution	Silicon dioxide [2] (SiO_2) dissolved	Aluminum oxide [2] (Al_2O_3) dissolved	Sulfur trioxide [2] (SO_3) dissolved
	Percent	*Percent*	*Percent*	*Percent*
1	42.6	0.45	0.10	2.38
2	36.7	.68	.10	1.92
3	42.1	.49	.10	2.48
4	33.8	.58	.11	1.33
5	37.2	.56	.11	1.66
6	46.1	.71	.06	1.92
7	29.3	1.39	.07	.93
8	30.0	.50	.14	1.10
9	37.5	.36	.17	1.21
10	45.2	.43	.10	2.09
11	28.7	2.59	.06	.47
12	27.3	.77	.13	.38
13	22.5	1.74	.08	.22
14	29.0	.51	.16	.80
15	33.8	.76	.09	.91
16	38.1	.26	.41	1.58
17	38.7	.27	.25	.95
18	23.9	.29	.34	.49
19	24.2	2.47	.06	.32
20	25.0	.72	.12	.27
21	22.2	2.26	.06	.22
22	35.7	.84	.08	.86
23	44.3	.76	.06	1.54
24	18.2	.58	.19	.30
25	41.4	.96	.05	.76
26	27.9	3.26	.05	.35
27	50.7	.66	.04	2.06
28	31.5	3.32	.05	.35
29	25.8	.48	.20	.50
30	28.7	1.04	.08	.70
31	31.8	.34	.19	2.02
32	18.2	.83	.14	.17
33	34.1	1.52	.06	1.31
34	44.9	1.33	.03	1.50

[1] Fly ash treated with 1 N. solution of sodium hydroxide for 24 hours at 80° C. in a sealed container. ASTM Method C 289-52T, except that a 12.5-gram sample was used.

[2] Expressed as a percentage of the original weight of fly ash.

Table 15.—Solubility of alkalies in water after various periods of aging fly ash and lime slurries

Fly ash number	Proportion [1] of sodium oxide soluble at—			Proportion [1] of potassium oxide soluble at—			Proportion [1] of equivalent alkalies as sodium oxide soluble at—			Equivalent alkalies as sodium oxide [2]	
	7 days	28 days	91 days	7 days	28 days	91 days	7 days	28 days	91 days	Total amount	Soluble at 28 days
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
1	18	42	42	12	34	40	15	38	41	3.21	1.23
2	19	35	46	12	30	42	16	33	44	3.55	1.16
3	25	43	41	13	33	37	20	39	39	3.55	1.38
4	14	29	31	12	37	57	12	35	52	1.61	.56
5	19	42	45	12	39	47	15	40	46	2.79	1.13
6	22	44	50	18	49	66	19	47	62	1.82	.86
7	20	40	40	14	48	59	15	46	55	1.78	.82
8	13	31	37	10	34	41	11	33	39	2.64	.88
9	15	45	55	10	42	61	12	44	58	1.59	.70
10	20	42	45	19	52	64	19	49	59	1.72	.85
11	12	42	35	10	48	52	10	46	48	1.72	.80
12	12	30	40	9	33	45	10	32	44	1.85	.60
13	14	36	59	9	41	54	10	40	55	1.40	.56
14	13	38	47	9	38	45	11	38	46	1.93	.74
15	13	45	68	10	45	70	12	45	69	1.67	.75
16	13	38	43	9	39	45	11	39	45	2.38	.92
17	12	24	48	15	44	53	14	37	51	1.87	.69
18	14	28	45	12	34	50	12	32	48	2.15	.69
19	8	36	56	9	46	66	9	44	64	1.57	.69
20	7	27	54	7	32	49	7	31	50	1.78	.55
21	8	27	37	8	37	46	8	35	44	2.29	.79
22	25	28	50	10	43	59	13	40	57	1.62	.64
23	13	33	60	9	36	65	11	35	63	1.77	.62
24	13	30	42	10	34	46	11	32	44	1.60	.52
25	16	34	47	20	47	56	19	45	54	1.64	.74
26	12	33	52	9	42	64	10	40	60	1.97	.78
27	21	48	48	22	60	63	22	56	59	1.76	.98
28	12	36	50	10	44	57	11	52	55	1.85	.97
29	30	38	62	7	25	39	15	29	47	1.84	.53
30	18	37	37	16	44	38	17	38	37	1.94	.74
31	24	46	55	12	37	48	16	41	50	3.11	1.26
32	9	34	41	8	36	48	8	35	46	1.79	.63
33	20	48	64	12	51	63	14	50	63	1.39	.70
34	14	46	54	13	51	59	13	49	58	1.88	.92

[1] Expressed as a percentage of the amount of the constituent present in the original fly ash sample (table 1, p. 122).

[2] Expressed as a percentage of the original weight of fly ash.

Specifications for fly ash generally include some requirement based on a test for strength of mortar prepared with cement or lime in combination with fly ash. When the mortar is prepared with lime, a minimum strength is usually specified. When cement is used, the strength of mortar containing fly ash is required to be equal to or greater than a definite percentage of the strength of a control mortar not containing fly ash. Any type I cement of acceptable quality is generally permitted in these tests.

It has been shown in this investigation that the strength ratio for a particular fly ash may vary considerably when determined with different cements. Consequently, when a minimum strength ratio is specified, consideration should be given to the cements to be used in the testing procedure. The most desirable procedure would be to use cement from a single source, or if this is not feasible, to use cements which have been found to produce comparable results in mortars containing fly ash. Further research on this problem is desirable to determine whether certain properties of cement can be used for selecting such cements. There is evidence, for example, that the alkali and tricalcium silicate contents of the cement may have an important bearing on its behavior with fly ash.

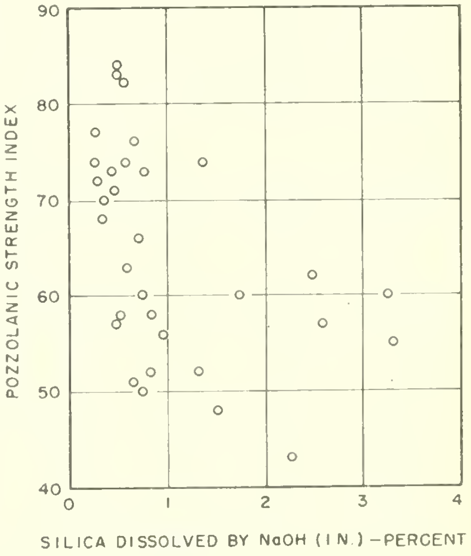

Figure 17.—Comparison of silica dissolved in NaOH solution and pozzolanic strength index.

Table 16.—Silica dissolved from fly ash after various periods of aging fly ash and lime slurries

Fly ash number	Acid soluble [1] at—			Alkali-acid soluble [2] at—		
	7 days	28 days	91 days	7 days	28 days	91 days
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
1	0.44	0.74	0.44	1.68	2.40	---
2	.41	.57	.41	2.03	1.56	---
3	.41	.73	.45	1.75	2.71	---
4	.29	.60	.35	.97	1.54	---
5	.37	.62	.38	1.77	1.60	---
6	.38	.63	.40	.71	1.93	---
7	.35	.55	.34	1.21	1.71	---
8	.32	.77	.39	2.07	2.38	---
9	.85	.47	.47	1.84	2.45	---
10	.40	.58	.44	1.38	1.69	---
11	.36	.68	.42	1.25	1.86	---
12	.41	.70	1.07	.72	2.59	0.92
13	.38	.67	1.03	.91	1.41	.97
14	.40	.73	.97	1.50	2.15	1.18
15	.40	.79	1.00	1.54	2.64	.95
16	.42	.64	.99	2.06	2.60	1.33
17	.39	.75	1.01	1.80	2.73	1.27
18	.41	.39	.64	1.78	2.57	1.14
19	.37	.30	.57	.74	2.46	1.31
20	.37	.34	.49	.87	2.26	1.10
21	.40	.30	.59	1.00	2.22	1.76
22	.34	.33	---	1.00	1.24	---
23	.40	.36	.63	.90	1.45	1.87
24	.44	.30	.57	1.11	1.32	.88
25	.44	.32	.53	1.53	1.85	1.32
26	.43	.33	.50	1.41	2.17	1.32
27	.38	.33	.53	1.38	1.97	1.19
28	.42	.32	.51	1.04	2.16	1.49
29	.44	.42	.51	1.63	1.96	.95
30	.46	.38	.42	1.73	1.82	1.25
31	.38	.29	.44	1.85	2.16	1.54
32	.46	.32	.49	.96	1.25	.98
33	.42	.30	.48	.85	1.53	.80
34	.41	.36	.43	1.08	2.05	1.11

[1] Amount dissolved by digesting the residue from water extraction in HCl (1:10) overnight. Expressed as a percentage of the original weight of fly ash.

[2] Amount dissolved by digesting the residue from the acid treatment for 30 minutes on steam bath in 1 percent NaOH solution. HCl added to make solution acid immediately before filtering. Results are expressed as percentages of the original weight of fly ash.

Table 17.—Alumina dissolved from fly ash after various periods of aging fly ash and lime slurries

Fly ash number	Acid soluble [1] at—			Alkali-acid soluble [2] at—		
	7 days	28 days	91 days	7 days	28 days	91 days
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
1	3.8	7.2	8.4	0.33	0.38	---
2	3.9	7.0	8.7	.36	.36	---
3	3.9	7.8	9.4	.48	.53	---
4	2.2	5.6	7.9	.46	.31	---
5	3.6	7.9	9.2	.38	.32	---
6	3.7	5.9	7.1	.19	.37	---
7	2.0	5.7	6.9	.28	.36	---
8	4.1	8.5	10.1	.51	.62	---
9	4.9	8.2	10.4	.44	.42	---
10	2.9	6.2	7.8	.29	.29	---
11	2.2	5.8	7.7	.26	.35	---
12	3.0	7.1	9.9	.32	.38	0.24
13	2.2	5.6	7.7	.19	.38	.25
14	4.1	7.5	9.5	.30	.36	.27
15	3.8	6.8	9.2	.19	.31	.21
16	5.2	9.4	11.0	.43	.44	.38
17	4.9	9.5	12.3	.51	.47	.36
18	4.9	9.9	13.0	.45	.53	.36
19	1.9	5.4	7.4	.12	.32	.23
20	1.9	5.1	8.2	.15	.25	.16
21	1.5	4.6	6.5	.18	.36	.32
22	2.1	5.3	---	.20	.16	---
23	2.1	4.8	7.4	.16	.21	.27
24	3.2	6.9	9.9	.36	.23	.25
25	3.7	6.1	7.6	.38	.31	.29
26	2.0	5.3	7.3	.35	.32	.31
27	2.7	6.0	7.2	.35	.28	.26
28	2.0	5.4	7.4	.20	.28	.31
29	4.0	7.5	11.2	.44	.44	.32
30	3.6	5.0	5.5	.15	.15	.20
31	3.6	7.2	9.5	.45	.41	.38
32	2.3	5.7	8.2	.19	.19	.21
33	2.0	5.3	7.8	.21	.21	.11
34	3.0	6.2	7.9	.32	.27	.19

[1] Amount dissolved by digesting the residue from water extraction in HCl (1:10) overnight. Expressed as a percentage of the original weight of fly ash.

[2] Amount dissolved by digesting the residue from the acid treatment for 30 minutes on steam bath in 1 percent NaOH solution. HCl added to make solution acid immediately before filtering. Results are expressed as percentages of the original weight of fly ash.

Table 18.—Iron oxide dissolved from fly ash after various periods of aging fly ash and lime slurries

Fly ash number	Acid soluble [1] at—			Alkali-acid soluble [2] at—		
	7 days	28 days	91 days	7 days	28 days	91 days
	Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
1	1.5	2.4	2.9	0.09	0.09	---
2	1.6	2.5	3.2	.08	.12	---
3	2.1	3.1	3.7	.13	.16	---
4	1.2	2.1	2.6	.05	.10	---
5	1.5	2.8	3.6	.10	.09	---
6	.9	1.6	1.9	.04	.10	---
7	.8	1.7	2.3	.06	.12	---
8	1.8	3.1	3.9	.15	.23	---
9	1.8	2.3	3.2	.10	.17	---
10	1.1	2.0	2.7	.07	.10	---
11	.7	1.6	2.3	.06	.09	---
12	.8	1.7	2.4	.08	.10	0.08
13	.6	1.3	2.0	.07	.10	.13
14	1.3	1.7	2.8	.07	.12	.17
15	1.1	1.6	2.6	.04	.10	.15
16	2.2	3.2	3.9	.09	.16	.21
17	2.0	3.2	4.5	.16	.16	.17
18	1.4	1.9	2.5	.07	.10	.08
19	.7	1.6	2.3	.06	.10	.14
20	1.4	2.7	4.3	.09	.14	.12
21	.6	1.2	1.5	.06	.09	.08
22	1.4	2.5	---	.10	.08	---
23	1.0	1.6	2.1	.06	.06	.37
24	.8	1.7	2.4	.09	.07	.11
25	1.3	2.1	2.8	.12	.12	.15
26	.7	1.7	2.7	.09	.08	.15
27	1.0	1.8	2.1	.15	.12	.17
28	.6	1.5	2.1	.07	.07	.15
29	1.0	1.9	2.0	.09	.10	.12
30	1.9	2.1	2.3	.06	.07	.11
31	1.8	2.9	3.9	.12	.16	.24
32	.7	1.6	3.8	.07	.08	.14
33	1.0	2.0	2.4	.15	.17	.20
34	1.0	1.8	2.6	.17	.21	.21

[1] Amount dissolved by digesting the residue from water extraction in HCl (1:10) overnight. Expressed as a percentage of the original weight of fly ash.

[2] Amount dissolved by digesting the residue from the acid treatment for 30 minutes on steam bath in 1 percent NaOH solution. HCl added to make solution acid immediately before filtering. Results are expressed as percentages of the original weight of fly ash.

The minimum strength ratio or pozzolanic strength index which should be specified depends on factors not considered in this study. The pozzolanic strength indexes of four fly ashes used in this series representing sources of fly ash which have given good results in concrete vary from 71 to 83. The feasibility of using fly ashes of poorer quality will depend on such factors as the permissible reduction which can be tolerated in the 28-day strength of the concrete and the quality of fly ashes available in a particular locality.

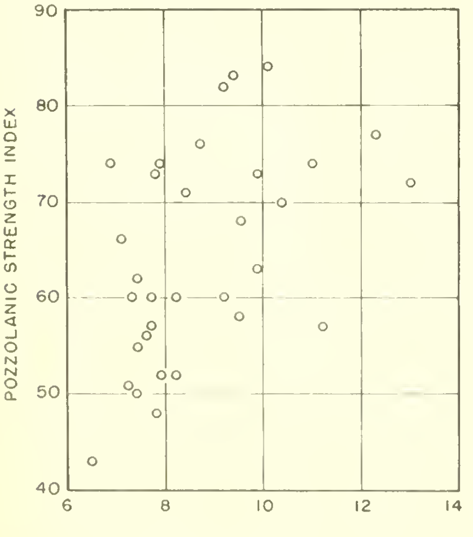

Figure 18.—Comparison of acid-soluble alumina after storage in lime solution for 91 days and pozzolanic strength index.

In the use of a lime-fly ash mortar for determining the pozzolanic activity of fly ash, control of the uniformity of the lime does not appear to be difficult. However, the results of the tests of fly ash with lime did not show as close a correlation with those in which cement was used as might be desired. This may have been due to the particular procedure followed in preparing the lime-fly ash mortars, and further work with this method may furnish better results.

At present the fineness of fly ash is usually specified in terms of specific surface determined by the air permeability method. The results given in this study show that specific surface so determined has no relation to the effect of fly ash on the strength of mortar. This is in contrast to the good correlation found with the amount passing the No. 325 sieve. The use of this sieve as a means for determining the fineness of fly ash appears to be desirable.

Except for carbon, the variations in the total amounts of individual chemical constituents that are present in the samples tested in this investigation had little or no relation to the pozzolanic activity of the fly ash. This does not necessarily mean that chemical constitution is not of importance, but rather than the normal variation in composition of the different fly ashes is not sufficient to produce significant changes in pozzolanic activity. It is likely that availability for reaction or the physical state of the constituents control the activity of a fly ash to a much greater extent

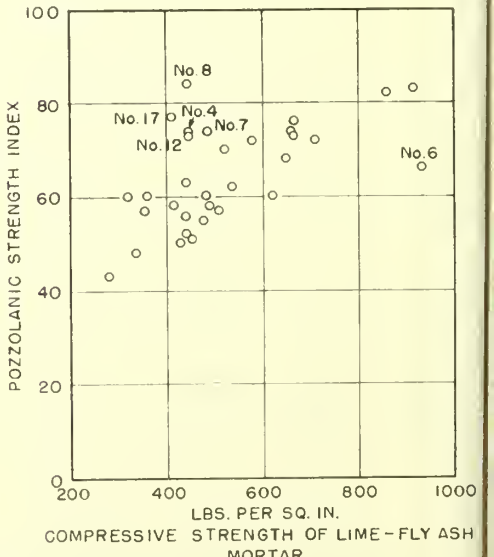

Figure 19.—Comparison of compressive strength of lime-fly ash mortar cured at 130° F. and pozzolanic strength index.

Table 19.—Results of lime-fly ash tests for pozzolanic activity

Fly ash number	Compressive strength at 7 days of 1:2:9 lime-fly ash mortar cured at—		Time of setting of 1:4 lime-fly ash mixture		Lime absorption at 28 days less that at 7 days, CaO per 100 grams of fly ash
	100° F.	130° F.	Initial	Final	
	P. s. i.	*P. s. i.*	*Hours*	*Hours*	*Grams*
1	670	580	140	313	2.25
2	785	670	141	195	1.39
3	925	920	52	120	1.98
4	175	445	234	258	.90
5	545	860	167	192	2.17
6	445	935	235	322	3.11
7	325	485	177	266	.60
8	385	445	90	168	−.23
9	175	520	120	195	.20
10	490	665	216	340	3.97
11	115	510	119	293	−.03
12	195	445	118	315	2.31
13	95	320	340	384	−.17
14	160	490	141	374	.32
15	115	485	142	194	1.49
16	235	660	119	143	1.96
17	105	410	72	651	−.04
18	160	710	166	450	.05
19	65	535	507	555	.00
20	65	360	187	404	----------
21	80	280	260	527	1.68
22	105	415	165	395	.86
23	135	425	315	482	1.75
24	100	440	170	315	−.01
25	210	440	139	420	3.47
26	125	620	339	651	.09
27	275	455	355	479	6.06
28	130	480	216	384	1.80
29	105	355	118	315	−.17
31	515	650	180	215	.01
32	110	440	340	501	−.28
33	55	335	259	404	−.16
34	175	440	381	673	2.83

than the total amount of constituent present. For this reason specification requirements for chemical constituents other than carbon are necessary only to limit potentially deleterious constituents or to eliminate freak materials. For fly ash, these requirements are generally limited to the percentages of silica, magnesia, sulfur trioxide, and in special cases available alkalies.

The ASTM Specification for fly ash (C 350–54 T) includes a minimum requirement for silica of 40 percent. Of the fly ashes examined in this investigation, six would fail to meet this requirement. Three of these six have pozzolanic strength indexes greater than 70 percent and appear satisfactory for use as admixtures in concrete. In view of this, it is believed that a slightly lower minimum silica content may be tolerated. A value of 35 percent is suggested as being suitable.

Limits for magnesia and sulfur trioxide are often included in a fly ash specification as a protection against possible deleterious effects in the concrete or mortar. This limit is usually 3 percent in each case. The maximum magnesium content found in any fly ash examined was 2.7 percent for sample 30, which was discarded for use in the mortar tests because of its coarseness and general unsuitability for use. The remaining samples showed a maximum value of 1.4 percent. Although a retention of the limit on magnesia to prevent the inclusion of unusual materials is believed necessary, these results indicate that the magnesia content for most fly ashes is well below this limit. The sulfur trioxide content, a measure of the amount of sulfates present, varied from 0.2 to 2.8 percent in the samples tested. A limit of 3 percent for this material appears to be suitable.

The alkalies present in a fly ash must be considered whenever the material is used in concrete in which there is a possibility that reactive aggregate may be present. The ASTM specification for fly ash (C 350–54 T) includes an optional clause which limits the available alkalies to 1.5 percent. As previously stated, these available alkalies are determined on a lime-fly ash slurry after storage at 100° F. for 28 days. All of the fly ashes included in this investigation meet this requirement, the largest amount of available alkalies being 1.38 percent for sample 3. However, it was also shown that in many cases the alkalies continued to dissolve after the 28-day period. It is believed that additional studies should be made to show the correlation between the results of this test and the amounts of alkalies becoming soluble in an actual mortar or concrete, and whether this continued solubility is of significance to the expansion resulting from the alkali-aggregate reaction.

Table 20.—Relation between limiting values of pozzolanic strength index and other properties of fly ashes

Number of fly ashes meeting indicated limits	Pozzolanic strength index, minimum	Carbon content, maximum	Loss on ignition, maximum	Amount passing No. 325 sieve, minimum	Average water requirement ratio, 35-percent replacement, maximum
		Percent	*Percent*	*Percent*	*Percent*
3	82	3.5	4.1	93.9	98
5	76	6.4	6.9	91.5	100
13	70	6.9	7.5	88.9	101
15	66	13.6	14.8	88.9	101
21	60	13.6	14.8	75.8	109
27	55	13.6	14.8	74.8	110
31	50	15.6	18.0	74.8	125
33	43	15.6	18.0	56.2	125

Appendix A

The following are general descriptions or details of testing procedures used in this investigation which may be of interest to those actively engaged in making similar studies.

Procedure 1

Methods of chemical analysis.—The oxide analyses, except for the alkalies, were conducted following the general methods outlined by Hillebrand, Lundell, Bright, and Hoffman (*10*) using a sodium carbonate fusion. The alkalies were determined by means of a flame photometer using the method described by Halstead and Chaiken (*11*).

Carbon was determined by weighing the CO_2 formed by ignition of the sample at 950° C. in a tube combustion furnace using oxygen. The total CO_2 absorbed was corrected for the amount resulting from carbonates by determining the CO_2 released by hydrochloric acid in a separate determination.

The sulfates (as SO_3) were determined in a portion of the sample dissolved in HCl(1:1) after 15–20 minutes digestion at a near boiling temperature.

Procedure 2

Preparation and analysis of lime-fly ash slurries.—The slurries were prepared as directed in the method for the determination of available alkalies as given in the Tentative Methods of Sampling and Testing Fly Ash for Use as an Admixture in Portland Cement Concrete, ASTM Designation C 311–54 T.

The extraction of the water soluble alkalies and the analyses of the residues were conducted as follows:

The liquid contents of the vial were transferred to a 250-ml. beaker and the solid cake transferred to a mortar. The cake was ground with the addition of some water until a uniform slurry was formed and no large lumps remained. The slurry was then transferred to the same 250-ml. beaker and sufficient water added to make the total volume 100 ml.

For most samples after 28 and 91 days, the solid cakes were so hard that the vial had to be broken and the glass carefully removed before grinding. The glass was scraped as thoroughly as possible to remove the adhering fly ash and lime.

After standing for 2 hours at room temperature with frequent stirring, the slurry was filtered and the residue washed thoroughly with hot water (8–10 times).

The filtrate was neutralized with HCl using 1 drop of phenolphthalein as indicator. It was then transferred to a 250-ml. volumetric flask and filled to the mark. This solution (or aliquot where necessary) was used to determine the amounts of sodium and potassium present by means of a flame photometer.

The flame photometer was calibrated with neutral solutions of sodium and potassium chlorides. The necessary correction for lime was made by determining the average amount present in several representative samples and using a correction curve established on that basis.

The residue from the water extraction was returned to the 250-ml. beaker and 13 ml. of concentrated HCl and 87 ml. of H_2O added. This amount was calculated to give a final concentration of free HCl equivalent to a dilution ratio of 1:10.

The beaker and its contents were allowed to stand overnight after frequent stirring for the first several hours. The liquid was filtered into a 250-ml. flask and the residue washed thoroughly with hot water. After the liquid had cooled, the flask was filled to the mark and aliquots of this solution were used for the determination of the acid soluble constituents.

The residue from the acid treatment was transferred to the 250-ml. beaker, and 100 ml. of hot 1-percent solution of sodium hydroxide added. It was then digested on a steam bath for 20 minutes. Five ml. of concentrated HCl were added to make the solution slightly acid and to dissolve any iron hydroxide or aluminum hydroxide which may have been released from glassy complexes by the alkali treatment.

The solution was filtered into a 250-ml. volumetric flask, and the residue washed thoroughly with hot water. After the filtrate had cooled the flask was filled to the mark and aliquots of this solution were used for the determination of the constituents soluble in or "solubilized" by the alkali treatment.

Photometric methods were used to determine the amounts of dissolved constituents in each solution. Silica was determined by the molybdenum blue method, and iron and aluminum oxides were determined with ferron. The procedures followed were essentially the same as described by Shapiro and Brannock (*12*), except for necessary modifications to adjust for acid concentration.

Procedure 3

Compressive strength of lime-fly ash specimens.—The lime-fly ash mortar used for these specimens contained by weight one part hydrated lime, two parts fly ash, and nine parts of graded Ottawa sand. Sufficient water was added to this mixture to produce a flow of 80 plus or minus 5 percent when determined as prescribed in the Standard Method of Test for Compressive Strength of Hydraulic Cement Mortars, AASHO Designation T 106–49. The materials were mixed by hand in a 6-quart bowl. Sufficient mortar was prepared to fill two 1⅞-inch diameter cylindrical glass tubes, 12 inches in length, which had previously been sealed at the bottom. These tubes were filled in 6 equal layers, each layer being tamped 25 times with a metal tamper having a plane face one inch in diameter.

Immediately after molding, the specimens were sealed on the top with a rosin-paraffin mixture. After storage at 73° F. for 24 hours, one specimen was stored for 6 days at 100° F. and one was stored at 130° F. Three hours before testing at 7 days, each specimen was cut into three equal cylinders, the glass stripped, and the cylinders capped with a sulfur capping compound. The capped specimens were kept in a moist condition at 73° F. until tested in compression.

Procedure 4

Time of setting of lime-fly ash specimens.—A paste consisting of one part of hydrated lime and four parts of fly ash by weight was prepared with sufficient water to produce a paste of normal consistency as determined by the Method of Test for Normal Consistency of Hydraulic Cement, ASTM Designation C 187–49. This paste was placed in a cylindrical container of about 3-inch diameter, and covered with a layer of saturated lime water, which in turn was covered with a film of lubricating oil to prevent carbonization.

At intermittent intervals, the paste was tested by means of the Vicat needle to determine when initial and final set occurred. Initial set was considered to have occurred when the 1-mm. needle penetrated to 35 mm. from the upper surface of the paste in 30 seconds. When the needle showed no penetration of the surface of the paste, final set was considered to have taken place.

Procedure 5

Lime absorption test.—This test is intended to determine the capacity of a fly ash for reacting with lime in a lime water solution. The following procedure was used:

A 1-gram sample of fly ash was placed in a test tube to which was added 100 ml. of a lime water solution of known concentration as determined by titration with a standardized N/25 hydrochloric acid solution. The test tube was sealed and agitated constantly by mechanical means until the date of test. At ages of 7 and 28 days, a 25-ml. portion of the supernatant liquid was removed and immediately titrated with the N/25 hydrochloric acid solution. After the sample for test at an age of 7 days had been taken, the solution in the test tube was replenished with 25 ml. of lime water of the concentration originally used.

The amount of lime absorbed by the fly ash was determined by the change in alkalinity of the solution during each period of agitation, as indicated by the amount of N/25 hydrochloric acid needed for titration.

Appendix B

The following discussion[5] is based on a number of published references, and while there is not always complete agreement on all details, it is believed that it represents generally the consensus of thought concerning the chemical constitution of the inorganic matter in coal and the changes which occur during combustion to form fly ash.

The bulk of the inorganic material in coal is derived from detrital or adventitious matter which fills the fissures and cracks occurring in coal beds. This material consists mainly of silicates, sulfides, and carbonates. The siliceous matter is mostly shale and clay, kaolin being the most abundant mineral present. Smaller amounts of potash clays, micas, feldspars, and quartz may also be present. The shales are largely complex alumino-silicates in combination with small amounts of iron, calcium, sodium, and potassium. The sulfide minerals present are essentially in the form of pyrite (FeS_2) and marcasite (FeS_2). Calcite ($CaCO_3$) and siderite ($FeCO_3$) make up the bulk of the carbonate minerals occurring in coal beds. It has been estimated that more than 90 percent of the mineral matter in coal consists of kaolin, calcite, pyrite, and clay.

The grinding of pulverized fuel largely separates the coal from the associated minerals and even the minerals from each other into discrete particles. As the particles are burned in an air suspension, collisions are infrequent and the minerals are not again mixed. Thus, their residues appear separately in the fly ash.

When bituminous coals are used as the pulverized fuel, the small particles of coal in suspension are heated rapidly. The particles soften and swell into multicellular bubbles of coke, known as "cenospheres." Some particles of bituminous coal may increase in volume by as much as 40 times shortly after entering the furnace, the average being about 11 times the original volume. The density of coal (1.3 to 1.7) is thereby decreased to approximately 0.12 to 0.15 by cenosphere formation. Much of the inorganic matter in coal is finely divided. The swelling of the coal further separates and subdivides the ash into minute particles. As the cenospheres burn, numerous bits of mineral ash are released.

The mineral matter undergoes considerable decomposition and chemical change during combustion, and the chemical nature of the ash is quite different from that of the original mineral matter in coal.

Each of the three main minerals—kaolin, calcite, and pyrite—break down to their respective oxides at the temperature of the fuel bed (980–1650° C.), but the resulting oxides usually have melting points above the combustion temperature. However, eutectic relationships are formed which result in fusion.

In the flame, the shale and clay particles lose combined water and those particles which leave the flame prematurely do not complete the processes of fusion or that of moisture evolution. These particles are white and rounded, the whiteness being due to cavities formerly occupied by steam. The particles which continue in the flame tend to complete both processes, particularly in relatively hot combustion chambers, and they form mostly complex silicates that are colorless, solid spheres resembling glass. Various tints of color may be due to iron components in the shale. Some crystalline aluminum silicate (mullite) is also found.

The pyrite (FeS_2) is ignited to the oxide state with the accompanying evolution of sulfur dioxide (SO_2) and sulfur trioxide (SO_3). If the temperature is below 1,000° C., ferric oxide (Fe_2O_3) forms. However, the most likely situation is that ferroso-ferric oxide or magnetite (Fe_3O_4) is formed, since temperatures usually exceed 1,000° C. This material is in the form of black, rounded or spherical particles which are often mistaken for particles of carbon. The magnetite particles can be identified by their magnetic properties.

The calcite, of course, breaks down to calcium oxide and carbon dioxide during ignition. Usually the bulk of free lime so produced combines with the sulfur dioxide and trioxide evolved from the pyrite to form calcium sulfate. Generally, the sulfur oxides are fixed as sulfates by any free particles of basic oxides. Free lime not so converted to sulfates will remain as calcium oxide or be converted to calcium hydroxide during storage.

The unburned coal particles which rise to the stack will be found in the fly ash as large, gritty, porous, irregularly shaped particles of cindery coke.

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[5] This discussion was prepared by Bernard Chaiken, Chemist, Physical Research Branch, Bureau of Public Roads.

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New Publication

A new publication entitled *Standard Plans for Highway Bridge Superstructures* (1956 edition) is now available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at $1.75 per copy.

The 52-page book comprises 18 series of standard plans which furnish details for a variety of types of highway bridge superstructures. Entirely new in this edition are three series for pretensioned precast concrete and five series for structural timber. From the previous (1953) edition, with revised and modernized designs, are series for I-beams, riveted or welded plate girders, reinforced concrete slabs, T-beams, and box girders, precast concrete decks, and post-tensioned precast concrete I-sections.

The plans will serve as a useful guide to State, county, and local highway departments in the development of suitable and economical bridge designs for primary, secondary, and urban highways. The drawings give sufficiently complete information so that they approach contract drawings as nearly as practicable.

The designs are based on two widths of roadway: 24 feet with H15–44 live load and 28 feet with H20–S16–44 live load. The ranges in span lengths, from 11 feet for structural timber to 140 for deck plate girder, are based on the economy and suitability of each particular type of superstructure.

Use of Fly Ash in Concrete

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Reported [1] by ALBERT G. TIMMS and WILLIAM E. GRIEB,
Highway Research Engineers

The purpose of this investigation was to study the properties of concrete containing, as replacement for part of the cement, four fly ashes selected from two widely separated areas because they represented sources of fly ash which have a history of extensive use in concrete. From each source, fly ashes of widely different fineness were selected with the expectation that those with the maximum fineness would have the lowest carbon content. Concrete specimens containing the fly ashes were tested for strength at various ages, for shrinkage at various periods when stored in a dry atmosphere, for durability in freezing and thawing, and for resistance to attack by calcium chloride used for ice removal.

These tests indicate, in general, that at ages through 28 days both flexural and compressive strengths were lower than comparable concrete without fly ash. At one year the strengths of the concrete containing fly ash were about as high or higher than the plain concrete. Fly ash did not improve the resistance to freezing and thawing of non-air-entrained concrete. In air-entrained concrete some of the fly ashes gave satisfactory durability and some did not. The particular portland cement used was also a factor. For the most part, concrete containing fly ash showed less shrinkage upon drying than concrete without fly ash. All substitutions of fly ash for portland cement adversely affected the resistance of both air-entrained and non-air-entrained concrete to attack by calcium chloride solutions.

IN an effort to develop a more durable concrete, research engineers have been seeking a low-cost form of finely divided silica for adding to portland cement concrete, which would react with the alkalies and lime liberated by the cement in the hydration process. A material which appears to have the desired properties is fly ash. This is a waste product which results from the burning of pulverized coal and accumulates in large quantities at power plants. It has a high silica content and a fineness similar to cement.

Not only have claims been made concerning the suitability of fly ash for producing a more durable concrete, but the product has been proposed as an extender or replacement for portland cement. Impetus has been given to the use of fly ash for this purpose because of the shortage of cement during the past few years.

In this study, the properties of concrete containing four fly ashes used as a substitute for cement were investigated.

Conclusions

Tests in which fly ash was used as a replacement for part of the cement indicated the importance of the carbon content of the fly ash. In general, the higher the amount of carbon in the fly ash and the greater the percentage of fly ash used as a replacement for cement, the lower was the compressive and flexural strength at all ages.

The detailed conclusions are as follows:

1. The 7-day compressive strengths of concretes containing fly ash were lower than comparable concrete without fly ash. With a few exceptions, the same relation was found for the 28-day strengths. The lower strengths were always obtained with the concretes containing the fly ashes of highest carbon content. At 1 year the strength of the concrete containing fly ash of low carbon content was always higher than that of the concrete without fly ash, in some cases as much as 20 percent higher; and the strength of the concretes containing 16⅔ percent of fly ashes X and Y (high carbon content) equaled or exceeded those without fly ash. Only the concretes containing 33⅓ percent of fly ashes X and Y failed to meet or surpass the strengths of the control concrete at 1 year.

2. Much the same general relations hold for flexural strength as were obtained for compressive strength, except that at 1 year the flexural strength of all concrete with fly ash replacement was greater than that of concrete without fly ash.

3. Non-air-entrained concrete with and without fly ash had poor resistance to freezing and thawing.

4. Satisfactory durability was obtained with the air-entrained concretes containing cements A and B without fly ash. For air-entrained mixes, similar results were found for the concretes containing cement B (low alkali content) and the various fly ash replacements. Satisfactory durability was also obtained with the concretes containing cement A (high alkali content) and the 16⅔-percent replacement of the four fly ashes and the 33⅓-percent replacement of fly ashes A and B; concrete with 33⅓-percent replacement of cement with fly ashes X and Y (high carbon content) had poor durability as indicated by factors of 40 or lower.

5. The effect on durability and strength of concrete containing fly ash from a given source varies with the characteristics of the cement used. Therefore, cement-fly ash combinations for which no service record is available should be investigated prior to use.

6. In general, there was little difference in the shrinkage upon drying of the concrete made with or without fly-ash substitutions.

7. Non-air-entrained concrete with or without fly ash was severely attacked by calcium chloride used for ice removal. The two air-entrained concretes without fly ash had good resistance to attack by calcium chloride, but all substitutions of fly ash for portland cement lowered the resistance of the concrete.

8. The amount of air-entraining material necessary to produce a given amount of air in concrete increases with the carbon content of the fly ash.

Scope of Tests and Materials Used

The fly ashes [2] from one area are identified hereafter as A and B, and from the other area as X and Y. From each source, fly ashes of widely different fineness were selected.

In this investigation specimens for compressive and flexural strength, durability, and volume change tests were made from concretes without air and with 3 to 6 percent entrained air. Since fly ash is generally used as a replacement for part of the portland cement in paving or structural concrete, it seemed logical that the air content should be maintained within the usual limits of 3 to 6 percent.

In order to investigate the possibility of reaction of the fly ash with the alkalies in the cement, two portland cements were used: one of relatively high and the other of relatively low alkali content. The chemical analyses of the two portland cements shown in table 1 indicate that cement A had an alkali content of 1.09 percent which is higher than that permitted by AASHO specification M 85–53 for a low alkali cement, and that cement B had a very low alkali content, 0.14 percent.

Each of the four fly ashes was used as a replacement for portland cement in two percentages (16⅔ and 33⅓) for strength determinations and freezing and thawing and volume change measurements. For the concrete slabs which were stored outdoors and exposed to the action of calcium chloride, only a 33⅓-percent replacement was used. In each case the replacement of cement by fly ash was on the basis of solid volumes.

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[1] This article was presented at the 59th Annual Meeting of the American Society for Testing Materials, Atlantic City, N. J., June 17–22, 1956.

[2] Fly ashes A, B, X, and Y are identified as 1, 3, 14, and 29, respectively, in the article *Studies relating to the testing of fly ash for use in concrete*, which appears in this issue.

Concrete for compressive and flexural strength test specimens was made with 1½-inch maximum size crushed limestone and a siliceous sand with a fineness modulus of 2.75. The aggregates were of good quality and had a good service record for durability. Aggregates from the same sources were used in making the volume change and freezing and thawing specimens and the outdoor exposure slabs, except that the maximum size was limited to 1 inch. This maximum size was required because of the small size specimens.

Mix Proportions

The mix data for all concrete specimens are given in table 2. Mixes without fly ash were designed for a cement content of 6.0 sacks of cement per cubic yard and a slump of approximately 3.0 inches. Where fly ash was used, one or two sacks of cement per cubic yard of concrete were replaced by an equal solid volume of fly ash. Because of differences in specific gravities of the cements and the fly ashes, the weight of fly ash for a solid volume equivalent to the solid volume of a 94-pound sack of cement was 74 pounds for fly ashes A, B, and X, and 72 pounds for fly ash Y. The quantity of sand in air-entrained mixes was reduced to compensate for the volume of added air, thus maintaining a constant cement content. For all mixes with the same maximum size coarse aggregate, the actual weight of coarse aggregate per cubic yard of concrete was constant.

The water requirement for concrete of a given slump varied with the amount and properties of the fly ash used for replacement of cement. The mix data for the strength-test specimens shown in table 2 indicate that when 16⅔ percent of fly ash A or B was used, the water was approximately 1¼ gallons per cubic yard of concrete less than that in the mixes without fly ash; and when 33⅓ percent of fly ash A or B was used, the reduction was approximately 2¼ gallons per cubic yard. When either 16⅔ or 33⅓ percent of fly ash X was used, the water requirement was the same as for concrete without fly ash, but when 16⅔ percent of fly ash Y was used, approximately ½ gallon more water was required. In the case of fly ash Y with 33⅓ percent replacement, approximately 1 gallon more water was needed. The same trends were noted for the concrete used in the freezing and thawing and volume change tests.

The quantity of air-entraining admixture required for a constant air content increased with a higher carbon content in the fly ash and with the quantity of fly ash used as a replacement for the portland cement. This variation is shown in table 2. When either 16⅔ or 33⅓ percent of fly ash A was used, approximately 10 percent less air-entraining agent was required than was needed for the concrete with 100 percent portland cement. Also when 33⅓ percent of fly ash Y was used with cement B, approximately 5 times as much air-entraining admixture was required. Slump and air content were maintained approximately constant for all concretes.

Table 1.—Chemical composition and physical properties of portland cements and fly ashes

	Portland cement		Fly ash			
	A	B	A	B	X	Y
Chemical composition (in percent):						
Silicon dioxide	22.0	22.3	47.1	49.2	41.2	38.5
Aluminum oxide	5.6	5.4	18.2	19.9	22.1	23.5
Ferric oxide	2.5	2.4	19.2	16.2	20.6	18.8
Calcium oxide	62.8	66.1	7.0	5.5	6.0	3.2
Magnesium oxide	3.0	1.0	1.1	1.4	1.2	1.0
Sulfur trioxide	2.0	1.7	2.8	2.7	.9	.6
Loss on ignition	.8	1.2	[1] 1.2	[1] 1.2	[1] 5.4	[1] 11.6
Sodium oxide	.40	.04	1.80	2.00	1.00	.60
Potassium oxide	1.05	.15	2.15	2.35	1.42	1.88
Total equivalent alkalies as Na_2O	1.09	.14	3.21	3.55	1.93	1.84
Insoluble residue	.16	.12	--------	--------	--------	--------
Chloroform-soluble organic substances	.009	.003	--------	--------	--------	--------
Free calcium oxide	.85	.56	--------	--------	--------	--------
Water-soluble alkali:						
Na_2O	.11	.01	--------	--------	--------	--------
K_2O	.63	.02	--------	--------	--------	--------
Computed compound composition (in percent):						
Tricalcium silicate	42	55	--------	--------	--------	--------
Dicalcium silicate	31	22	--------	--------	--------	--------
Tricalcium aluminate	11	10	--------	--------	--------	--------
Tetracalcium aluminoferrite	8	7	--------	--------	--------	--------
Calcium sulfate	3.4	2.9	--------	--------	--------	--------
Carbon	--------	--------	.2	.6	5.0	11.2
Carbon dioxide	--------	--------	.01	.04	.04	.03
Physical properties:						
Apparent specific gravity	3.20	3.17	2.49	2.52	2.51	2.43
Specific surface (Wagner) cm.^{2}/g	1,800	1,625	--------	--------	--------	--------
Specific surface (Blaine) cm.^{2}/g	--------	--------	3,075	4,305	2,565	3,220
Passing No. 325 sieve percent	--------	--------	93.2	95.2	84.5	89.4
Autoclave expansion do	.32	.04	--------	--------	--------	--------
Normal consistency do	25.5	25.0	--------	--------	--------	--------
Time of setting (Gillmore test):						
Initial hours	3.2	4.2	--------	--------	--------	--------
Final do	5.2	6.4	--------	--------	--------	--------
Compressive strength (1:2.75 mortar):						
At 7 days p. s. i.	2,340	2,960	--------	--------	--------	--------
At 28 days do	3,670	5,070	--------	--------	--------	--------
Mortar air content percent	9.1	6.8	--------	--------	--------	--------

[1] Determination made at 600° C.

Fabrication of Specimens

All mixing was done in an open pan-type mixer of 1¾ cubic foot capacity. The following mixing cycle was employed: Cement plus the fly ash and damp sand were mixed for 30 seconds, after which the estimated total quantity of mixing water plus the air-entraining admixture, if any, was added and the mortar mixed for a further period of 1 minute; coarse aggregate in a saturated surface-dry condition was then added and the concrete mixed for an additional 2 minutes, making a total mixing time of 3½ minutes. Air contents for the air-entrained concrete were determined by the pressure method, ASTM C 231–49 T, and for the non-air-entrained concrete it was calculated by the gravimetric method, ASTM C 138–44. Consistency was controlled by means of the slump test.

Testing Procedures

Strength specimens were molded, cured, and tested in accordance with applicable ASTM procedures. All strength specimens were moist cured continuously until tested.

The freezing and thawing tests were made on prisms 3 by 4 by 16 inches, sawed from 4- by 16- by 24-inch slabs. The concrete slabs were cured in the moist room for 14 days, stored in laboratory air for 7 days, and then sawed into 7 prisms measuring 3 by 4 by 16 inches. Three were used for the freezing and thawing test and the other four were held in moist air storage for future tests. The freezing and thawing prisms were returned to the moist room for 7 days, then immersed in water for 7 days, and at a total age of 35 days the freezing and thawing cycle was started.

A 24-hour cycle was used in making the freezing and thawing tests. Each specimen was frozen and thawed in water in an individual metal container of such size as to provide about ⅛-inch clearance on all sides. The specimens in the containers were placed vertically in the freezing chamber. The refrigerant used was alcohol and it entirely surrounded the sides of the container to a height of about 18 inches. The specimens in the containers were frozen at −10° F. for 18 hours, then removed from the freezer and thawed in a water bath at 70° F. for 6 hours. After each cycle, the specimens were removed from the containers, turned upside down, and replaced. The water in the containers was changed.

Resistance to the effects of alternate freezing and thawing was determined periodically during the test by measuring changes in the dynamic modulus of elasticity of the concrete. For a given specimen, changes in dynamic modulus of elasticity E are proportional to changes in the square of the natural frequency of vibration (N^2). It is therefore possible to use the reduction in N^2 directly as a measure of the deterioration of the concrete. Freezing and thawing was continued until the specimens showed a loss in N^2 of 40 percent or for 200 cycles, whichever occurred first.

In addition to the 4- by 16- by 24-inch slabs which were sawed for the freezing and thawing prisms, six 3- by 4- by 16-inch beams were made for volume change tests. These specimens were removed from the molds after 24 hours under moist burlap and then stored in laboratory air at 72° F. and 50 percent relative humidity. After 180 days in laboratory

Table 2.—Concrete mix data

Fly ash			Mixes for strength tests [2]							Mixes for freezing and thawing and volume change tests [5]					
Identification	Source	Amount of replacement [1]	Cement	Fly ash	Water	Slump	Weight of plastic concrete	Air [3]	Admixture [4]	Cement	Fly ash	Water	Slump	Weight of plastic concrete	Air [3]
PORTLAND CEMENT A: NON-AIR-ENTRAINED CONCRETE															
		Percent	*Sack/cu. yd.*	*Lb./cu. yd.*	*Gal./cu. yd.*	*Inches*	*Lb./cu. ft.*	*Percent*	*Percent*	*Sack/cu. yd.*	*Lb./cu. yd.*	*Gal./cu. yd.*	*Inches*	*Lb./cu. ft.*	*Percent*
None	--------	------	6.0	--------	33.7	3.0	151.6	1.6	------	5.9	--------	35.5	2.7	148.6	2.0
A	Midwest	16.7	5.0	74	32.9	3.1	151.6	1.5	------	5.0	74	33.0	2.5	149.1	2.8
A	do	33.3	4.1	148	31.0	2.6	151.4	1.7	------	4.0	148	34.2	2.5	148.9	1.4
B	do	16.7	5.1	74	31.8	2.6	152.0	1.5	------	5.0	74	34.8	2.4	149.4	1.3
B	do	33.3	4.1	148	31.1	2.7	152.1	1.3	------	4.0	148	35.1	3.3	149.3	.8
X	East	16.7	5.0	74	33.5	3.1	151.4	1.4	------	5.0	74	35.6	3.0	149.6	1.0
X	do	33.3	4.0	148	33.7	3.2	150.9	1.2	------	4.0	148	35.0	4.0	148.4	1.5
Y	do	16.7	5.0	72	33.6	2.7	151.2	1.4	------	4.9	72	36.1	3.5	148.6	1.4
Y	do	33.3	4.0	144	34.2	3.2	150.2	1.4	------	4.0	144	35.2	3.7	147.7	1.7
PORTLAND CEMENT A: AIR-ENTRAINED CONCRETE															
None	--------	------	6.0	--------	30.4	3.0	147.3	4.6	100	5.8	--------	31.9	3.0	141.2	6.6
A	Midwest	16.7	5.0	74	29.4	3.0	146.6	4.9	93	4.8	74	31.4	3.5	141.0	6.2
A	do	33.3	4.1	148	28.7	3.2	146.6	4.7	87	4.0	148	31.3	3.0	144.1	4.5
B	do	16.7	5.0	74	29.2	3.0	146.4	4.8	137	5.0	74	31.0	3.0	145.0	4.2
B	do	33.3	4.1	148	28.8	3.1	147.5	4.3	152	4.0	148	31.9	3.0	145.1	3.9
X	East	16.7	5.0	74	30.5	2.9	146.7	4.3	155	4.9	74	32.8	3.3	143.0	5.0
X	do	33.3	4.0	148	30.8	3.2	146.2	4.2	217	4.0	148	34.8	3.0	144.4	4.0
Y	do	16.7	5.0	72	31.0	2.8	145.5	4.8	242	4.8	72	34.5	4.0	141.6	5.8
Y	do	33.3	4.0	144	32.2	3.3	144.7	4.4	395	4.0	144	36.0	3.5	144.7	3.1
PORTLAND CEMENT B: NON-AIR-ENTRAINED CONCRETE															
None	--------	------	6.0	--------	34.8	3.0	151.6	1.3	------	5.9	--------	36.9	3.2	149.2	1.3
A	Midwest	16.7	5.0	74	33.3	3.1	151.4	1.4	------	4.9	74	36.9	3.0	149.3	.7
A	do	33.3	4.0	148	31.9	3.1	151.2	1.6	------	4.0	148	35.6	3.5	149.2	.7
B	do	16.7	5.0	74	32.8	2.9	151.6	1.5	------	4.9	74	35.8	3.5	148.9	1.3
B	do	33.3	4.0	148	31.6	3.0	151.6	1.6	------	4.0	148	34.3	3.5	149.2	1.1
X	East	16.7	5.0	74	34.6	3.5	150.9	1.3	------	4.9	74	37.2	3.0	148.5	1.1
X	do	33.3	4.0	148	34.5	3.5	150.5	1.2	------	4.0	148	37.0	3.2	148.4	.8
Y	do	16.7	5.0	72	34.7	3.2	150.8	1.3	------	4.9	72	36.3	2.7	148.6	1.3
Y	do	33.3	4.0	144	35.1	2.7	149.7	1.4	------	3.9	144	36.6	3.0	147.2	1.6
PORTLAND CEMENT B: AIR-ENTRAINED CONCRETE															
None	--------	------	6.0	--------	31.6	2.7	146.0	4.6	100	5.9	--------	32.7	2.8	143.4	5.4
A	Midwest	16.7	5.0	74	30.9	3.1	146.4	4.4	89	4.9	74	32.8	3.5	144.1	4.9
A	do	33.3	4.0	148	29.5	3.2	146.0	4.8	93	3.9	148	30.5	3.2	142.4	5.6
B	do	16.7	5.0	74	30.5	3.0	146.2	4.5	143	4.9	74	31.8	3.0	143.9	4.7
B	do	33.3	4.1	148	29.5	3.1	146.5	4.4	179	4.0	148	30.6	3.0	142.8	5.2
X	East	16.7	5.0	74	31.5	3.1	145.1	4.9	189	4.8	74	33.3	3.0	141.8	5.6
X	do	33.3	4.0	148	31.5	3.5	144.4	4.8	261	3.9	148	34.0	4.3	140.7	5.6
Y	do	16.7	5.0	72	32.5	3.1	146.4	4.2	279	4.9	72	35.0	3.3	144.1	3.1
Y	do	33.3	4.0	144	33.1	3.7	144.8	4.3	489	3.9	144	35.1	4.0	143.7	3.7

[1] Fly ash used as replacement for cement on a solid volume basis.
[2] Proportions for mixes without fly ash were as follows: For non-air-entrained concrete, 94–190–350; for air-entrained concrete, 94–170–330. Maximum size coarse aggregate, 1½ inches.
[3] Air contents for non-air-entrained concrete were calculated, whereas air contents for air-entrained concrete were determined by pressure method (ASTM method C 231-49 T).
[4] Relative amount of air-entraining admixture used to produce air in concrete for strength tests; amount used in mixes without fly ash considered as 100 percent.
[5] Proportions for mixes without fly ash were as follows: For non-air-entrained concrete, 94–205–330; for air-entrained concrete, 94–188–330. Maximum size coarse aggregate, 1 inch.

air, they were stored in water for 14 days and then returned to laboratory air for an additional 180 days. The length of each specimen was determined when it was taken out of the mold and periodically thereafter.

Additional 4- by 16- by 24-inch slabs were made later to determine the effect of fly ash on the resistance of concrete to scaling due to freezing and ice removal with calcium chloride. A raised edge was cast around the perimeter of each slab. In these tests only concretes containing 33⅓-percent replacement of each of the four fly ashes for both cements were used with the non-air-entrained and the air-entrained mixes. The slabs were cast in June 1952 and were stored in moist air for 30 days. They were then placed outdoors on the ground. During the fall and winter season the top surface of each slab was kept covered with water. When ice was frozen on the slabs calcium chloride was applied to the surface at the rate of 2.4 pounds per square yard. After the ice was completely thawed the surface was washed and fresh water left on the surface to await another freezing. A total of 80 cycles were obtained during three winter seasons: 1952–53, 17 cycles; 1953–54, 34 cycles; and 1954–55, 29 cycles. The slabs were rated periodically for surface scale.

Discussion of Test Results

The results of the tests for strength, volume change, and durability are shown in tables 3–5 and in figures 1–5. The data are discussed in relation to the effect of substituting in two percentages each of the four fly ashes for a portion of each of the two portland cements.

Effect on strength.—The flexural and compressive strengths of the concretes without fly ash and with fly ash replacing part of each cement are shown in table 3. This table also shows the relative strengths of concretes containing fly ash expressed as percentages of the strengths developed by the concrete without fly ash. These relative strengths for various ages of test are shown graphically in figures 1 and 2.

In the upper diagram of figure 1 are shown the relative compressive strengths of the non-air-entrained concretes made with cement A and containing the substitutions of the two percentages of each of the four fly ashes. The 7-day strengths of concretes containing fly ash ranged from 53 to 94 percent of the strength of comparable concrete without fly ash. At 28 days the concretes containing fly ash A or B were equal to or greater than the 28-day strength of the concrete without fly ash. The concretes containing fly ash X or Y at 28 days ranged in relative strength from 74 to 96 percent.

All of the non-air-entrained concrete con-

Table 3.—Results of strength and freezing and thawing tests [1]

Fly ash		Air	Modulus of rupture			Compressive strength			Relative strength [2]						Air	Loss [3] in N² at—							Durability factor
Identification	Amount of replacement		7 days	28 days	1 year	7 days	28 days	1 year	Modulus of rupture			Compressive strength				10 cycles	25 cycles	50 cycles	100 cycles	150 cycles	175 cycles	200 cycles	
									7 days	28 days	1 year	7 days	28 days	1 year									
Portland Cement A: Non-Air-Entrained Concrete																							
	Pct.	Pct.	P. s. i.	P. s. i.	P. s. i.	P. s. i.	P. s. i.	P. s. i.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	Pct.	
None	----------	1.6	590	790	790	3,160	4,410	5,750	100	100	100	100	100	100	2.0	25	45 (24)	----------	----------	----------	----------	----------	6
A	16.7	1.5	525	740	835	2,930	4,480	6,300	89	94	106	93	102	110	2.8	22	54 (24)	----------	----------	----------	----------	----------	5
A	33.3	1.7	465	720	1,000	2,420	4,420	6,390	79	92	127	77	100	111	1.4	16	56 (24)	----------	----------	----------	----------	----------	5
B	16.7	1.5	550	790	955	2,970	4,640	6,640	93	100	121	94	105	115	1.3	24	57 (24)	----------	----------	----------	----------	----------	5
B	33.3	1.3	485	855	1,030	2,510	4,640	6,140	82	108	130	79	105	107	.8	16	48 (24)	----------	----------	----------	----------	----------	6
X	16.7	1.4	525	770	830	2,620	4,220	5,960	80	97	105	83	96	104	1.0	31	44 (14)	----------	----------	----------	----------	----------	4
X	33.3	1.2	360	665	880	1,900	3,350	5,770	61	84	111	60	76	100	1.5	41 (7)	----------	----------	----------	----------	----------	----------	2
Y	16.7	1.4	515	640	865	2,450	3,900	6,020	87	81	109	78	88	105	1.4	38 (7)	----------	----------	----------	----------	----------	----------	2
Y	33.3	1.4	355	610	835	1,690	3,270	4,970	60	77	106	53	74	86	1.7	51 (6)	----------	----------	----------	----------	----------	----------	1
Portland Cement A: Air-Entrained Concrete																							
None	----------	4.6	595	790	745	3,100	4,140	5,410	100	100	100	100	100	100	6.6	5	4	9	9	14	16	18	82
A	16.7	4.9	530	715	790	2,670	3,970	5,620	89	91	106	86	96	104	6.2	4	3	13	7	12	15	20	80
A	33.3	4.7	445	740	825	2,240	3,780	5,750	75	94	111	72	91	106	4.5	5	4	15	12	24	28	34	66
B	16.7	4.8	505	785	860	2,720	4,040	5,970	85	90	115	88	98	110	4.2	6	4	14	17	24	32	37	63
B	33.3	4.3	445	785	930	2,410	4,030	6,000	75	99	125	78	97	111	3.9	5	5	6	10	23	31	39	61
X	16.7	4.3	515	770	830	2,530	3,660	5,580	87	97	111	82	88	103	5.0	7	6	7	13	23	28	29	71
X	33.3	4.2	355	640	865	1,810	3,260	5,240	60	81	116	58	79	97	4.0	8	7	12	23	48 (150)	----------	----------	40
Y	16.7	4.8	470	660	760	2,230	3,490	5,410	79	84	102	72	84	100	5.8	5	5	6	6	14	18	19	81
Y	33.3	4.4	335	555	760	1,490	2,640	4,400	56	70	102	48	64	81	3.1	10	11	20	39 (86)	----------	----------	----------	25
Portland Cement B: Non-Air-Entrained Concrete																							
None	----------	1.3	705	880	850	3,770	5,890	7,480	100	100	100	100	100	100	1.3	4	9	42 (52)	----------	----------	----------	----------	15
A	16.7	1.4	640	790	935	3,150	5,220	8,690	91	90	110	84	80	116	.7	3	8	25	56 (85)	----------	----------	----------	19
A	33.3	1.6	535	815	1,025	2,380	4,460	8,250	76	93	121	63	76	110	.7	5	15	38 (52)	----------	----------	----------	----------	16
B	16.7	1.5	575	915	1,035	3,160	5,580	8,730	82	104	122	84	95	117	1.3	2	12	46 (48)	----------	----------	----------	----------	14
B	33.3	1.6	530	905	1,100	2,740	4,970	9,020	75	103	129	73	84	121	1.1	4	26	47 (34)	----------	----------	----------	----------	9
X	16.7	1.3	555	750	915	2,620	4,490	8,050	79	85	108	69	76	108	1.1	2	12	46 (48)	----------	----------	----------	----------	14
X	33.3	1.2	460	645	935	1,820	2,930	6,740	65	73	110	48	50	90	.8	9	36	44 (34)	----------	----------	----------	----------	9
Y	16.7	1.3	620	770	910	2,660	4,390	7,730	88	88	107	71	75	103	1.3	4	14	63 (48)	----------	----------	----------	----------	11
Y	33.3	1.4	425	690	995	1,750	3,130	6,570	60	78	117	46	53	88	1.6	36	51 (14)	----------	----------	----------	----------	----------	3
Portland Cement B: Air-Entrained Concrete																							
None	----------	4.6	700	775	750	3,840	5,480	6,270	100	100	100	100	100	100	5.4	2	4	4	4	9	8	9	91
A	16.7	4.4	650	730	860	3,320	4,800	7,260	93	94	115	86	88	116	4.9	1	3	5	1	9	7	14	86
A	33.3	4.8	495	750	900	2,590	4,200	7,110	71	97	120	67	77	113	5.6	0	3	1	2	8	7	7	93
B	16.7	4.5	600	750	875	3,450	5,290	7,540	86	97	117	90	97	120	4.7	0	5	1	5	6	11	14	86
B	33.3	4.4	570	805	875	2,850	4,510	7,440	81	104	117	74	82	119	5.2	0	7	2	7	7	8	9	91
X	16.7	4.9	575	735	825	2,860	4,490	6,710	82	95	110	74	82	107	5.6	0	4	2	8	6	7	9	91
X	33.3	4.8	445	635	835	1,970	3,310	5,700	64	82	111	51	60	91	5.6	0	2	4	6	11	14	16	84
Y	16.7	4.2	555	745	825	2,870	4,400	6,500	79	96	110	75	80	104	3.1	0	4	6	5	6	8	8	92
Y	33.3	4.3	370	605	780	1,840	2,870	5,420	53	78	104	48	52	86	3.7	0	4	8	7	15	16	16	84

[1] Each value for mixes without fly ash is an average of six tests; for mixes containing fly ash, each value is an average of three tests.
[2] Based on strength of concrete without fly ash.
[3] Figures in parentheses indicate number of cycles at which final reading was taken.

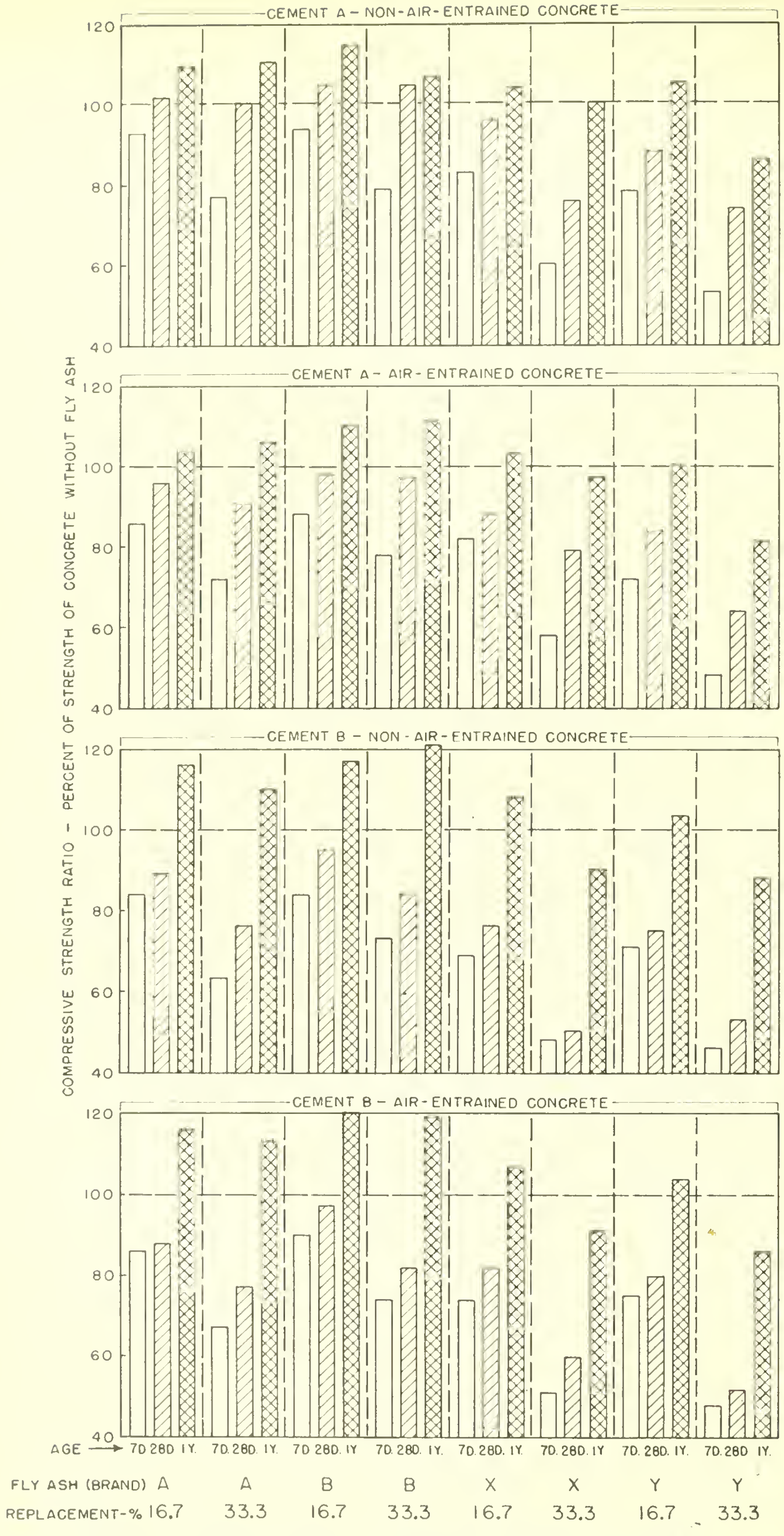

Figure 1.—Effect of fly ash on compressive strength of concrete.

taining cement A and fly ash equaled or exceeded the strength of comparable concrete without fly ash at 1 year, with the exception of that containing 33⅓ percent of fly ash Y. The concrete containing 33⅓ percent of fly ash Y had a relative strength of 86 percent of the comparable concrete without fly ash. The highest relative strength at 1 year for concrete made with cement A was 115 percent.

The three remaining diagrams in figure 1 show a pronounced similarity in the relative strength to that shown in the first diagram. This is also apparent in table 4 which shows the ranges and averages of relative strengths.

It is significant that the 7-day compressive strengths of the concretes containing fly ash are lower in each case than the comparable concrete without fly ash. With the exception of cement A, non-air-entrained concrete containing fly ash A or B, the same relation is found at 28 days. For the fly ashes of low carbon content, both replacements of cement with fly ash resulted in greater concrete strengths at 1 year than for concrete without fly ash. In one case, the increase in strength was 21 percent. Replacement of 16⅔ percent of the cement with fly ashes X and Y resulted in somewhat lower relative strengths at 1 year, but in all cases these were equal to or greater than 100 percent. Concretes containing 33⅓ percent of high carbon fly ashes (X or Y) had strengths less than the concrete without fly ash at all ages including 1 year, except fly ash X, used as replacement for cement A in non-air-entrained mixes; the latter had approximately the same strength at 1 year.

The use of air entrainment appeared to have little effect on the relative strengths, particularly with cement B. For each combination of cement and fly ash, the concrete with and without air entrainment had similar relative compressive strengths.

Figure 2 gives a comparison of the relative flexural strengths. The same general relation shown for compressive strengths exists for the flexural strengths, as is apparent in table 4, except that in no case did the concrete with the fly ash substitutions fail to equal the flexural strength of the concrete without fly ash at the 1-year period.

Durability.—The data from the freezing and thawing tests are shown in table 3 and in figures 3–4. In table 3 are shown the losses in natural frequency squared (N^2) at 10, 25, 50, 100, 150, 175, and 200 cycles and the durability factor.

The reductions in N^2 plotted against the number of cycles in figure 3 show that all the concretes containing fly ash and without air-entrainment deteriorated very rapidly under freezing and thawing. With two exceptions all air-entrained concrete with and without fly ash substitutions withstood 200 cycles with a reduction in N^2 of less than 40 percent. The exceptions were the concretes made from cement A and containing 33⅓ percent of fly ashes X and Y which had air contents of 4.0 and 3.1 percent, respectively.

In order to express in a single factor the loss in N^2 as related to the number of cycles of freezing and thawing required to produce that loss, a value known as the durability factor (*DF*) is used.[3] The values for durability factor shown in table 3 are calculated as follows:

$$DF = \frac{PN}{M}$$

Where:

DF = durability factor of the test specimens.
P = relative dynamic modulus of elasticity at N cycles in percent.
N = number of cycles at which P reached 60 percent or 200, whichever is less.
M = 200.

The durability factors and the air contents of the concrete are shown graphically in figure 4. To insure satisfactory resistance of the concrete to the alternate freezing and thawing in the type of freezing test used, the durability factor should be 50 or greater. Durability factors for the concretes without entrained air were all less than 20. All the concretes without air made with cement A had durability factors of 8 or less.

Of the 18 concretes in which air was entrained only 2 had durability factors of less than 60. The two concretes contained portland cement A and high carbon fly ashes: one with 33⅓ percent of fly ash X, and the other, 33⅓ percent of fly ash Y. The comparable concretes made with cement B had durability factors in excess of 80. For air-entrained concrete containing cement A with fly ash substitutions there appears to be a general relation between air content and the durability factor, the higher air content being associated with greater durability. This relation was not apparent for cement B.

It should be pointed out here that it was very difficult to obtain a uniform air content because concretes with the high carbon fly ashes required such large amounts of the air-entraining admixture to give the desired air content. Although the air content of the concrete in all cases was raised above 3 percent by the use of excessive quantities of the air-entraining admixture, the concrete containing fly ash X used as a replacement for 33⅓ percent of cement A with an air content of 4.0 percent had a durability factor of 40; concrete containing fly ash Y with 3.1 percent of entrained air had a durability factor of 25. In the concretes in which 16⅔ percent of cement A was replaced with fly ashes X and Y, the air contents were 5.0 and 5.8 percent and the durability factors were 71 and 81, respectively.

The durability factors for all air-entrained concretes made with cement B were 84 or above, even though the concrete with 16⅔ percent of fly ash Y had an air content of only 3.1 percent.

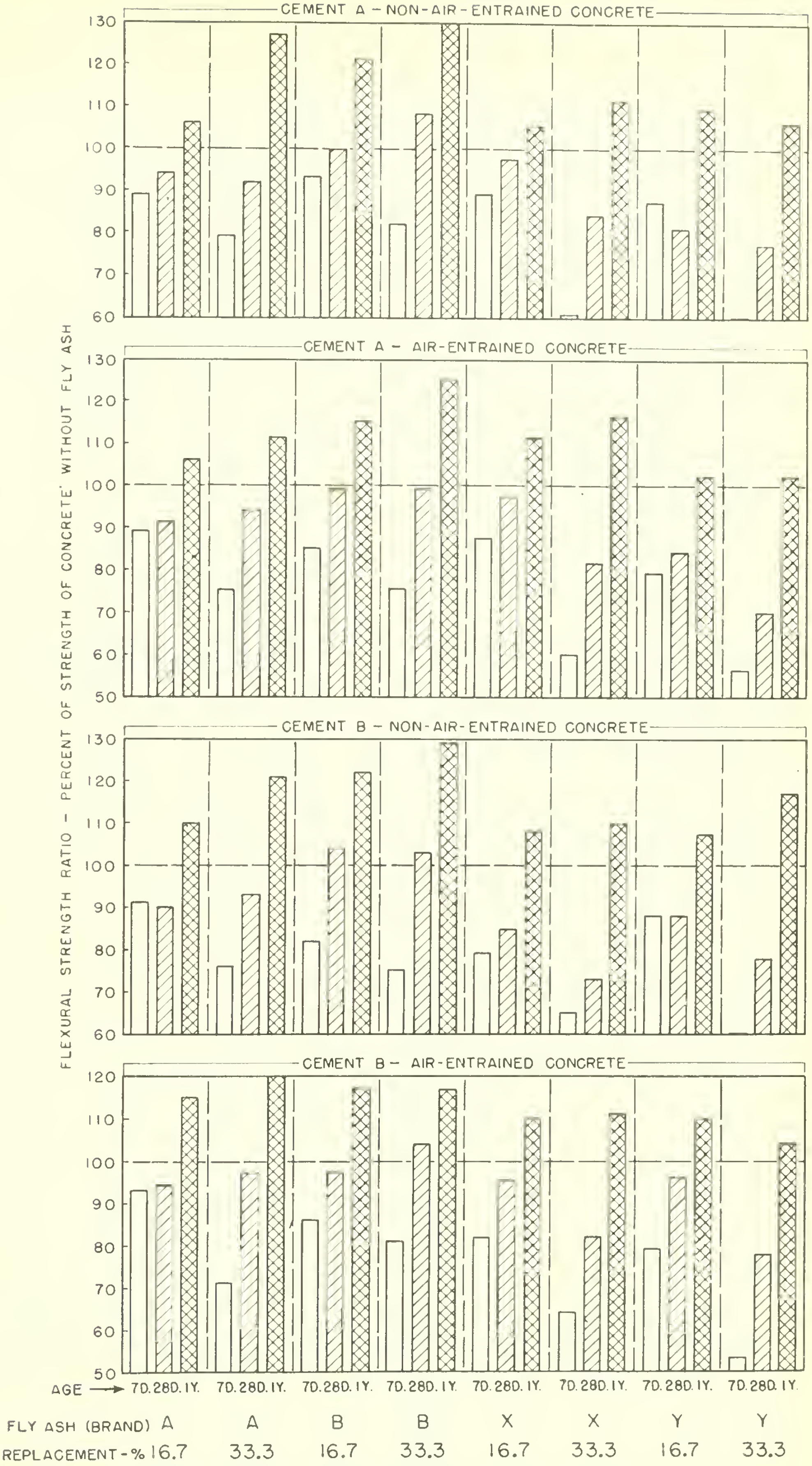

Figure 2.—Effect of fly ash on flexural strength of concrete.

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[3] See tentative method of test for *Resistance of concrete specimens to slow freezing and thawing in water or brine*, ASTM Designation C 292-52T. ASTM Standards 1955, part 3, pp. 1347-1350.

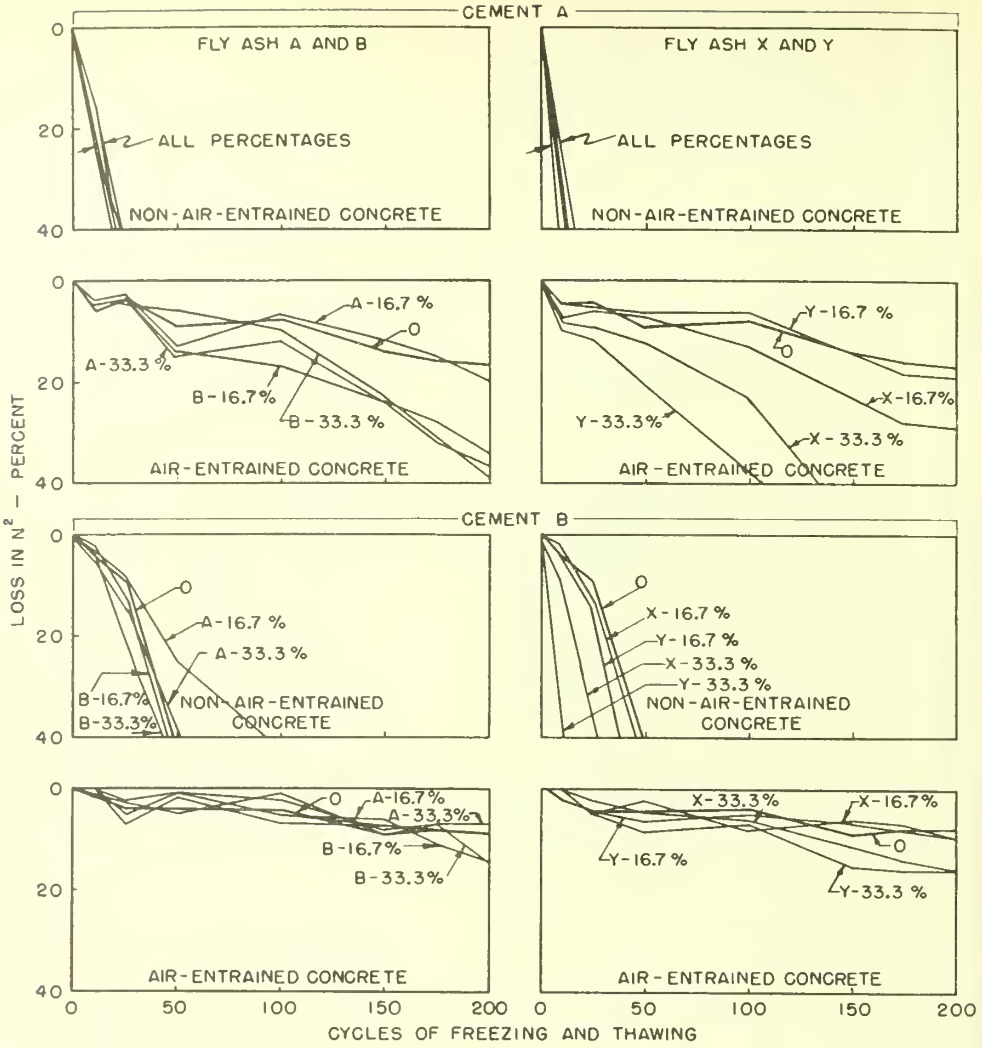

Figure 3 **(Right).**—*Effect of fly ash on resistance of concrete to freezing and thawing.*

The air-entrained concrete containing cement A without fly ash had a durability factor of 82, whereas the mix containing cement B had a factor of 91. These high factors indicate very little difference in durability between the two cements and offer no explanation as to why they act differently in concrete when fly ash is substituted for part of the cement.

Volume change.—The volume change data for all concretes are shown graphically in figure 5. The values plotted in the diagrams were actual test values. These curves show that there is very little difference in the volume change characteristics of the concrete made without fly ash substitutions and those containing various percentages of fly ash. Also, those containing the higher percentages of fly ash had less shrinkage in general than those with the smaller percentages of fly ash.

Resistance to scaling.—Calcium chloride, frequently used for thawing ice on concrete pavements, has been found to be very injurious to non-air-entrained concrete.[4] Air-entrained concrete usually offers good resistance to attack from chloride salts.

Table 5 shows periodic visual ratings of exposure slabs at 12, 17, 39, 51, and 80 cycles.

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[4] *Resistance of concrete surfaces to scaling action of ice-removal agents*, by Albert G. Timms, Highway Research Board Bulletin 128, 1956, pp. 20–50; also *Factors affecting resistance of portland cement concrete to scaling action of thawing agents*, by same author, PUBLIC ROADS, vol. 28, No. 7, Apr. 1955, pp. 143–157.

Figure 4 **(Below).**—*Effect of fly ash on durability of concrete.*

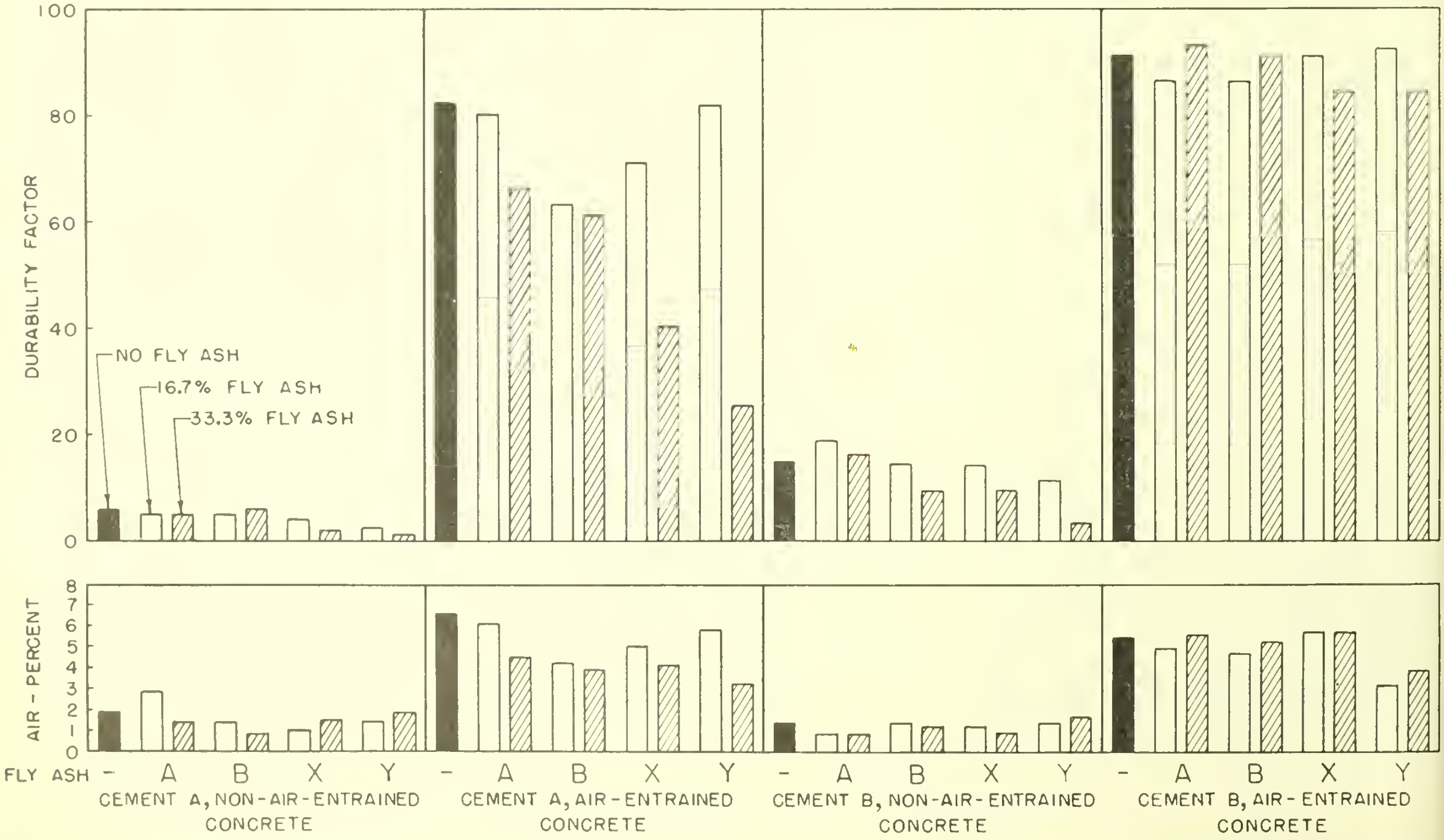

Table 4.—Relative compressive and flexural strengths of concrete containing fly ash

Cement	Type of concrete	Fly ash replacement	Relative compressive strengths at—						Relative flexural strengths at—					
			7 days		28 days		1 year		7 days		28 days		1 year	
			Range	Average	Range	Average	Range	Average	Range	Average	Range	Average	Range	Average
		Pct.	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
A	Non-air-entrained	16.7	94–78	87	105–88	98	115–104	108	93–87	90	100–81	93	121–105	110
	do	33.3	79–53	67	105–74	89	111–86	101	82–60	70	108–77	90	130–106	118
	Air-entrained	16.7	88–72	82	98–84	92	110–100	104	89–79	85	99–84	93	115–102	108
	do	33.3	78–48	64	97–64	83	111–81	99	75–56	66	99–70	86	125–102	113
B	Non-air-entrained	16.7	84–69	77	95–75	84	117–103	111	91–79	85	104–85	92	122–107	112
	do	33.3	73–46	58	84–50	66	121–88	102	76–60	69	103–73	87	129–110	119
	Air-entrained	16.7	90–74	81	97–80	87	120–104	112	93–79	85	97–94	96	117–110	113
	do	33.3	74–48	60	82–52	68	119–86	102	81–53	67	104–78	90	120–104	113

Table 5.—Visual ratings of surface scaling of concrete slabs [1] exposed to action of calcium chloride

Fly ash		Air [2]	Rating after exposure to outdoor freezing and thawing with CaCl₂ for—				
Identification	Amount of replacement		12 cycles	17 cycles	39 cycles	51 cycles	80 cycles
PORTLAND CEMENT A: NON-AIR-ENTRAINED CONCRETE							
	Percent	*Percent*					
None	------	1	5	8	10	10	10
A	33.3	1	6	8	10	10	10
B	33.3	1	6	8	10	10	10
X	33.3	1	7	8	10	10	10
Y	33.3	1	6	8	10	10	10
PORTLAND CEMENT A: AIR-ENTRAINED CONCRETE							
None	------	4.0	1	2	2	3	4
A	33.3	4.6	2	4	3	4	5
B	33.3	5.4	3	4	4	5	7
X	33.3	4.9	4	6	7	8	8
Y	33.3	4.3	4	6	6	8	8
PORTLAND CEMENT B: NON-AIR-ENTRAINED CONCRETE							
None	------	1	3	8	10	10	10
A	33.3	1	4	8	9	10	10
B	33.3	1	4	8	10	10	10
X	33.3	1	6	8	10	10	10
Y	33.3	1	5	8	10	10	10
PORTLAND CEMENT B: AIR-ENTRAINED CONCRETE							
None	------	6.8	1	2	2	2	3
A	33.3	4.0	1	2	4	4	5
B	33.3	3.3	5	6	7	8	8
X	33.3	3.9	6	8	8	8	8
Y	33.3	5.8	6	8	8	8	9

[1] Specimens were 4- by 16- by 24-inch slabs.
[2] Mix is same as used for freezing and thawing specimens except for air content.

The numerical significance of the ratings is as follows:

0—No scaling.
1—Very light spotted scale.
2—Spotted scale.
3—Light scale over about one-half the surface.
4—Light scale over most of surface.
5—Light scale over most of surface, few moderately deep spots.
6—Moderately deep scale spotted.
7—Moderately deep scale over one-half the surface.
8—Moderately deep scale over entire surface.
9—Deep scale spotted, otherwise moderate scale.
10—Deep scale over entire surface.

The concretes without air-entrainment and without fly ash had very poor resistance to attack by the chloride salt. None of the fly ashes used as replacements for part of the cement were effectual in improving the resistance of non-air-entrained concrete to attack by calcium chloride.

Entrained air greatly increased the resistance of the concrete without fly ash. In air-entrained concrete all the fly ash replacements for cement were detrimental to the resistance to scaling of the concrete, as indicated by the ratings of 3 to 8 at 39 cycles as compared with 2 for the concretes without fly ash. The extent of attack by calcium chloride did not appear to differ much with the brand of cement or with the fly ash used as a replacement.

It would appear from these data that if there is a possibility that calcium chloride might be used for ice removal, the use of fly ash as a substitute for portland cement in the concrete in the percentage used in this investigation is questionable.

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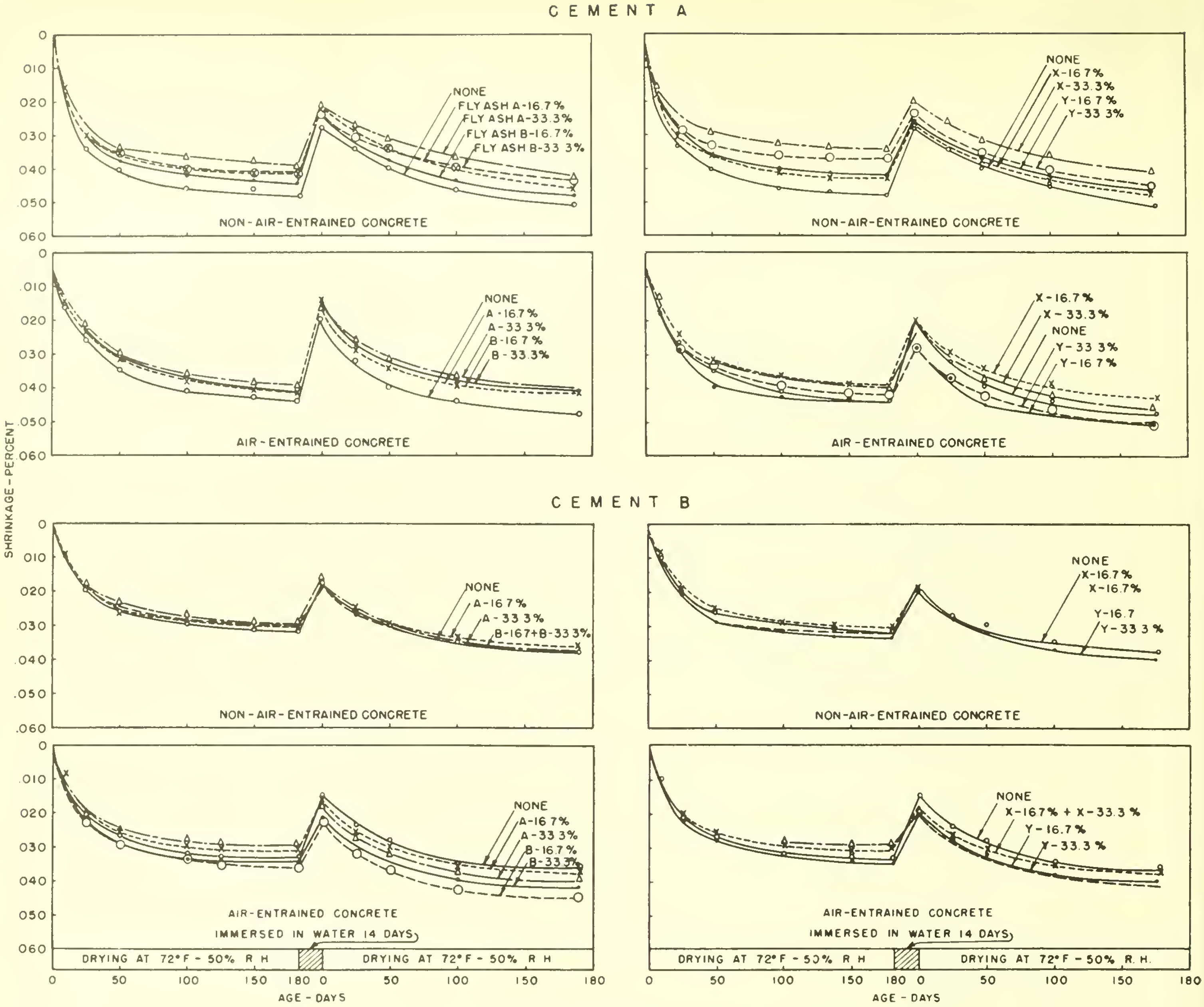

Figure 5.—Effect of fly ash on drying shrinkage of concrete.

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Labor Usage in Highway Construction as Influenced by Improved Equipment

BY THE CONSTRUCTION BRANCH
BUREAU OF PUBLIC ROADS

Reported by EDWIN J. COPPAGE, JR., Chief, Construction Management Section, and EDWIN L. STERN, Supervising Highway Engineer

LABOR USAGE DATA collected by the Bureau of Public Roads since 1944 indicate a pronounced decrease in the number of man-hours of direct labor required for a given physical volume of highway construction. Direct labor is defined as labor employed by the contractor at or near the site of the project. Indirect labor is that employed in the off-site production, manufacture or shipment of materials, supplies or equipment used, and labor induced by the respending of wages earned on the project.

The decrease in man-hour usage, which is illustrated in figures 1 and 2, is basically due to increased productivity of the labor-equipment combination resulting from great strides that have been made by manufacturers in developing more efficient construction equipment.

The curve in figure 1 shows the trend of man-hour usage per million dollars of construction cost from 1944 to 1955 and projected to 1970. The data for the plotted points are based on labor usage and contract costs reported by contractors for Federal-aid highway construction projects. The projected trend for the years 1955–70 was made by deriving an empirical formula for a smooth curve to fit the plotted points from 1944 to 1954. From a value of approximately 217,700 man-hours per million dollars construction cost in 1944, the factor had decreased 43 percent to about 123,000 in 1955. The projected curve indicates values of 104,000 for 1960 and 80,000 for 1970.

Since a million dollars purchased a different physical volume of work each year, due to varying price levels, the factors have been adjusted by means of the Bureau of Public

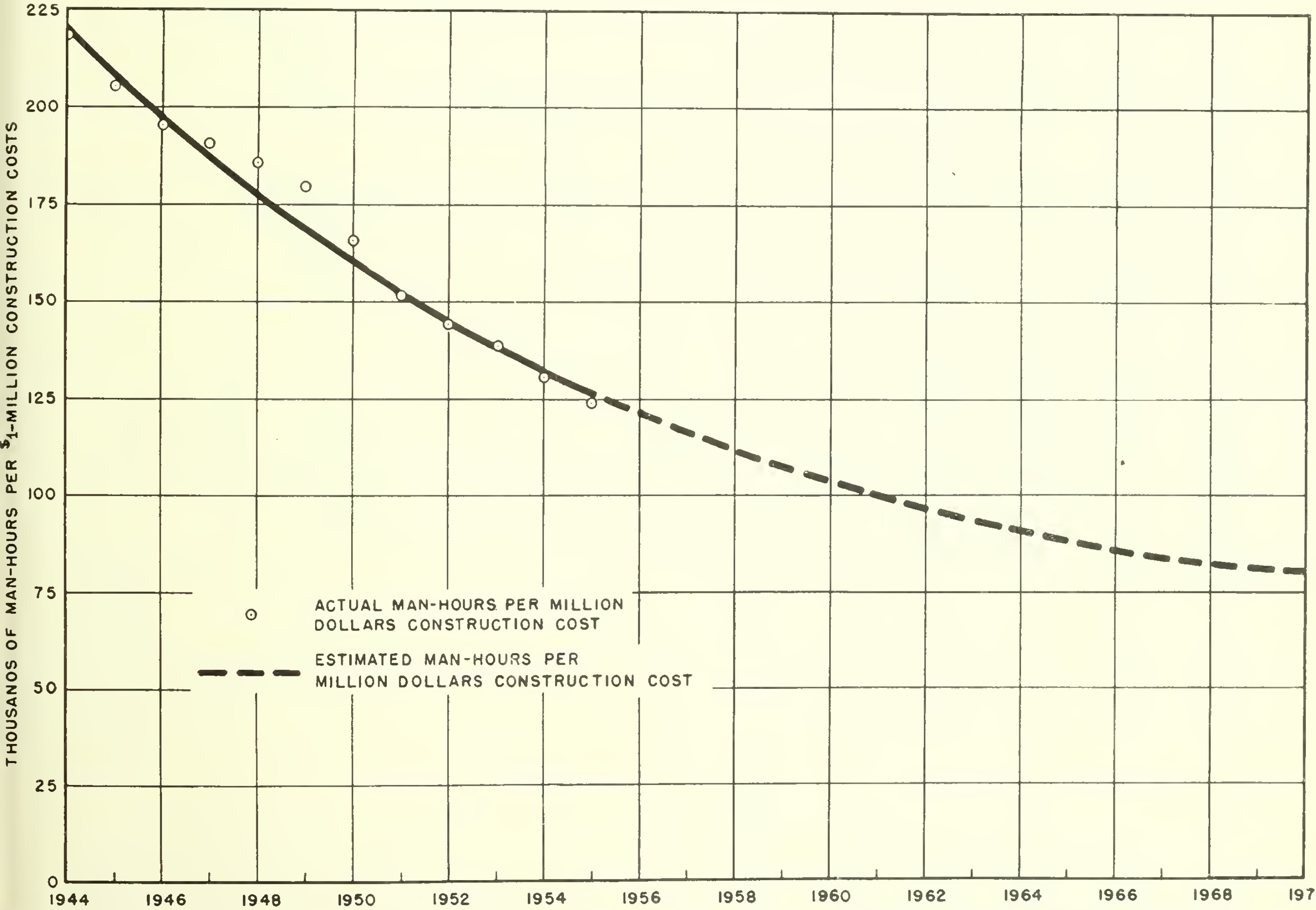

Figure 1.—Man-hours of labor required per million dollars of highway construction expenditures, adjusted to 1954 bid price level and excluding right-of-way and engineering costs.

Table 1.—Comparison between physical volume of all highway construction and highway construction labor usage

Calendar year	Physical volume		Labor usage	
	Construction expenditures [1]	Index, 1944=100	Man-hours	Index, 1944=100
	Billions		*Millions*	
1944	$0.448	100.0	97.6	100.0
1945	.459	102.5	94.3	96.5
1946	.986	220.1	193.4	198.1
1947	1.590	354.9	305.7	313.0
1948	1.823	406.9	342.4	350.6
1949	2.062	460.3	369.2	378.2
1950	2.263	505.1	374.8	383.8
1951	2.434	543.3	370.6	379.5
1952	2.594	579.0	374.9	384.0
1953	2.908	649.1	399.8	409.5
1954	3.659	816.7	474.5	486.0
1955	3.962	884.4	488.8	500.6

[1] Construction expenditures, adjusted to 1954 bid price level, exclude costs of right-of-way and engineering.

Roads highway construction price index to show for each year, the man-hours required to produce a physical volume of construction equivalent to that which could be purchased for a million dollars in 1954.

The practical application of the labor usage factors is found in estimating manpower requirements for various size highway construction programs and in evaluating changes in productivity and the effect of such changes on highway construction prices.

Factors Affecting Future Labor Requirements

The curve in figure 1 is plotted to follow the more significant known points, the trend of which is projected to represent future potentials. The actual factors, subsequent to 1955, may differ appreciably from the projected curve if there are changes in the trends of labor and equipment availability, in the trend of equipment improvement, and in the trend of wage rates. For example, in case of a scarcity of labor, it would be the tendency of contractors to increase their usage of labor-saving equipment, thus lowering the man-hour factor. If the trend in equipment improvement becomes more pronounced, a reduced labor factor will result. A pronounced increase in the wage-rate trend could have the effect of causing contractors to increase their usage of improved equipment, thereby lowering the man-hour usage factor. A decrease in the wage-rate trend could have the opposite effect.

The annual labor usage factors shown in figure 1 have been used in deriving the estimated labor requirements for all highway construction during the 13-year period, 1957–69. To bring all highway systems up to adequacy, as determined by the nationwide highway needs study published in 1955, 8.2 billion man-hours of labor would be required during the 13-year period, or an average annual labor force of about 396,000 workers, assuming 1,600 man-hours per year per worker.

Increased Productivity, 1944–55

Figure 2 and table 1 show the relation, each year, from 1944 through 1955, between the physical volume of all highway construction performed and the number of man-hours of labor used. These data indicate that the construction volume in 1955 was nearly nine times that for 1944, while labor usage in 1955 was five times that for 1944. The divergence between the two curves in figure 2 is indicative of increased productivity of the labor-equipment combination.

The 43-percent reduction in labor usage factors since 1944 is equivalent to a similar reduction in contractors' labor costs. During the period 1944–55, construction bid prices increased about 35 percent. Had there been no reduction in labor usage factors, the bid price increase during this period would have been approximately 61 percent.

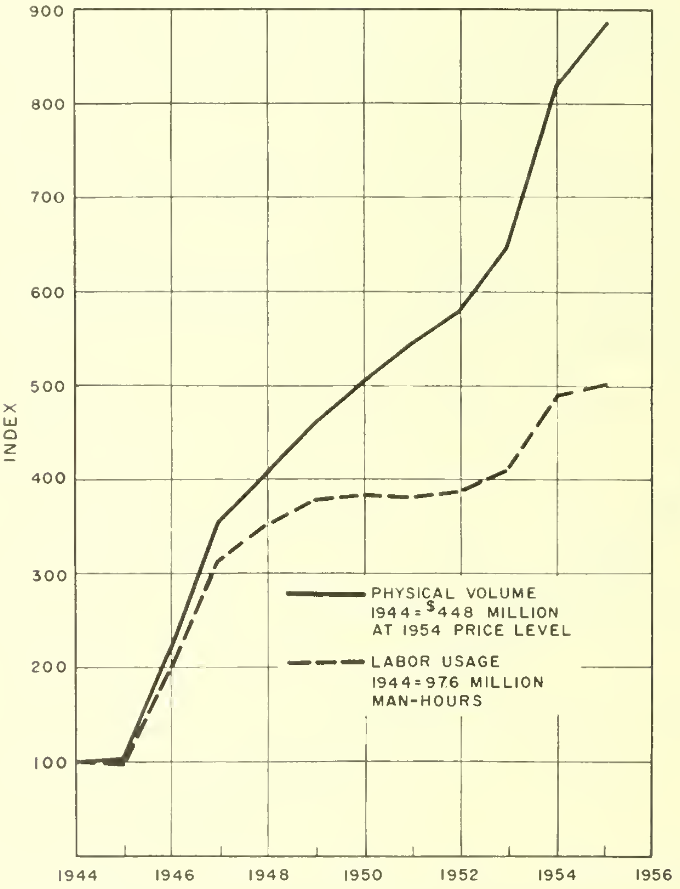

Figure 2.—Comparison between physical volume (excluding right-of-way and engineering costs) of all highway construction and highway construction labor usage.

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF DECEMBER 31, 1956

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$30,173	$40,511	$30,060	470.1	$10,255	$6,941	65.2	$61,876	$37,214	672.9	$112,642	$74,215	1,208.2
Arizona	31,274	10,690	8,521	155.6	5,839	4,260	41.9	10,414	7,966	100.5	26,943	20,747	298.0
Arkansas	41,965	19,868	11,264	494.9	8,418	4,488	77.4	29,016	18,149	316.1	57,302	33,901	888.4
California	77,938	45,461	29,729	225.8	40,166	30,022	55.3	283,150	138,558	238.8	368,777	198,309	519.9
Colorado	44,286	9,784	5,487	105.7	5,093	3,124	35.0	27,815	18,718	156.0	42,692	27,329	296.7
Connecticut	40,656	13,060	8,800	5.8	7,528	6,511	9.7	8,241	4,320	13.7	28,829	19,631	29.2
Delaware	23,009	1,870	935	24.9	832	449	9.6	10,153	5,128	47.8	12,855	6,512	82.3
Florida	48,114	14,034	7,450	248.8	13,547	6,935	54.9	53,319	30,692	325.5	80,900	45,077	629.2
Georgia	75,916	34,889	18,428	501.5	9,930	5,584	55.4	70,369	37,740	750.1	115,188	61,752	1,307.0
Idaho	35,658	3,332	2,182	41.9	3,706	2,407	31.5	13,490	9,194	164.3	20,528	13,783	237.7
Illinois	80,094	65,968	44,176	443.9	28,299	19,646	55.3	145,689	98,453	660.8	239,956	162,275	1,160.0
Indiana	98,335	24,747	13,386	97.0	6,247	3,464	48.5	36,861	20,888	147.8	67,855	37,738	293.3
Iowa	35,579	47,226	32,610	775.1	15,835	10,992	100.3	33,362	20,676	814.8	96,423	64,278	1,690.2
Kansas	27,938	39,376	32,851	655.4	5,284	2,798	62.9	39,148	22,982	1,036.6	83,808	58,631	1,754.9
Kentucky	64,981	7,247	4,113	95.5	343	190	1.3	39,455	23,311	306.8	47,045	27,614	403.6
Louisiana	47,968	39,066	18,603	145.3	7,853	4,148	9.9	37,455	20,084	254.5	84,374	42,835	409.7
Maine	28,682	7,097	3,832	42.5	1,525	917	9.1	16,139	8,436	99.0	24,761	13,185	150.6
Maryland	12,066	26,456	15,266	87.6	13,937	11,000	16.8	51,883	32,498	139.6	92,276	58,764	244.0
Massachusetts	43,141	38,536	22,829	31.6	38,226	23,362	26.7	50,811	28,437	47.2	127,573	74,628	105.5
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Nebraska	62,755	3,904	2,280	131.4	4,260	2,677	42.5	22,167	11,398	805.0	30,331	16,355	978.9
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New Jersey	88,545	12,556	6,278	62.8	5,219	2,608	1.7	36,313	18,154	47.2	54,088	27,040	111.7
New Mexico	14,717	21,006	18,486	93.0	5,650	4,959	39.6	20,940	15,238	195.7	47,596	38,683	328.3
New York	186,776	13,946	7,153	49.1	69,945	47,470	39.9	274,121	147,693	338.8	358,012	202,316	427.8
North Carolina	89,834	13,635	7,435	150.9	7,307	3,999	37.4	51,031	24,950	664.9	71,973	36,384	853.2
North Dakota	21,715	24,838	20,410	777.0	6,326	3,762	255.5	14,561	7,916	652.6	45,725	32,088	1,685.1
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Oregon	30,692	9,273	7,229	60.1	3,773	2,846	12.8	36,031	25,011	195.3	49,077	35,086	268.2
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South Dakota	26,938	27,605	18,809	576.5	2,723	1,524	104.3	16,308	9,586	556.9	46,636	29,919	1,237.7
Tennessee	50,630	27,507	13,142	572.2	13,969	6,985	56.8	78,484	47,806	403.2	119,960	67,933	1,032.2
Texas	155,635	20,328	10,403	356.3	45,542	33,271	262.6	131,490	75,028	1,354.0	197,360	118,702	1,972.9
Utah	21,643	18,325	15,168	137.0	817	638	1.9	8,153	6,320	134.0	27,295	22,126	272.9
Vermont	14,417	7,453	5,148	29.1	287	224		12,655	8,340	49.0	20,395	13,712	78.1
Virginia	68,806	20,286	11,873	142.7	6,699	3,705	19.4	34,413	17,879	327.5	61,398	33,457	489.6
Washington	54,821	5,845	3,819	73.0	6,925	4,181	66.1	31,051	18,036	196.1	43,821	26,036	335.2
West Virginia	46,589	16,124	8,485	47.3	6,255	3,140	24.3	23,457	11,878	72.5	45,836	23,503	144.1
Wisconsin	70,712	11,171	5,871	160.7	8,306	4,180	47.2	54,451	32,474	290.8	73,928	42,525	498.7
Wyoming	7,380	24,048	19,474	204.7	7,957	6,464	65.3	20,587	14,866	235.3	52,592	40,804	505.3
Hawaii	7,178	1,444	716	2.8	3,168	1,568	4.9	4,481	2,122	7.7	9,093	4,406	15.4
District of Columbia	22,554	11,581	7,281	4.2	842	437		9,208	5,853	.1	21,631	13,571	4.3
Puerto Rico	11,831	7,153	3,042	23.5				21,007	9,780	59.5	28,160	12,822	83.0
Alaska	15,074												
TOTAL	2,627,629	1,116,346	708,487	12,358.9	650,786	437,379	2,496.5	2,522,579	1,443,388	17,343.7	4,289,711	2,589,254	32,199.1

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C. M. Billingsley, *Editor*

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Financial Planning for an Expanded Highway Program

BY THE DIVISION OF FINANCIAL AND
ADMINISTRATIVE RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by G. P. St. CLAIR, Chief of Division,
and THOMAS R. TODD, Transportation Economist

Of the nationwide estimate of $101 billion in immediate highway needs, $45 billion were found on the Federal-aid systems below the level of the Interstate; and $33 billion on roads and streets not eligible for Federal aid. Engineering plans to meet these needs must be matched by equally adequate financial plans. A successful financial plan should (1) provide for completion of the accelerated highway improvement program within the desired number of years; (2) take care of maintenance, administration, and other regular commitments of the highway department; (3) meet interest and principal charges on the debt, if any, incurred in financing the program; and (4) provide sufficient additional revenues to meet the gradually increasing needs for replacement and expansion in the years following completion of the program.

A technique for the examination of alternate financial plans, familiarly known as the "cut-and-fill" method, was applied to the needs of the Federal-aid systems (other than Interstate) as they might be found to exist in an average State. The indications are that substantial increases in the rates of State taxes for highways must be brought about if these systems are to be improved to adequacy within a reasonable number of years. Financing with current revenues only would require drastic tax increases during the period of the accelerated program, with the prospect of a considerable reduction after its close. By resort to bond issues such a program may be financed by a relatively moderate increase of tax rates extending over the entire period of the bond issue. Although the problem will be found different in each State and the decisions made will be governed by prevailing fiscal policy, the procedures for critical study of different financing proposals are applicable in all cases.

RECENT STUDIES of highway needs have supplied the figure of $101 billion as the investment necessary for an adequate road and street plant. Not unnaturally the spotlight has been thrown upon the Federal aspects of the problem, and especially upon the financial requirements of the National System of Interstate and Defense Highways. It is the purpose of this article to examine the needs of the highway systems below the Interstate level, in an effort to gauge the nature and magnitude of the financing problem as it would confront the average State.

Needs of the Several Road and Street Systems

In figure 1 a perspective is given on the nationwide highway problem as the estimates were worked out in the highway needs study of 1954. Highway systems are grouped into three classes: the Interstate system, other highways eligible for Federal aid, and non-Federal-aid highways, chiefly county and local roads and streets. Of the $101 billion ($100 billion in continental United States) in estimated 10-year needs, the Interstate system, as it was constituted in 1954, claims slightly less than a quarter. The recent addition of 2,300 miles in urban areas will probably raise the $23 billion estimate by $4 or $5 billion, part of which will be a net addition to the grand total. The needs of other Federal-aid highways total nearly $45 billion—not far from half; and those of non-Federal-aid roads and streets, $33 billion, or about one-third.

A facet of the problem that has perhaps received too little attention is that of the requirements for replacement and for increased highway capacity after the completion of the catch-up program of accelerated highway construction. Roads—even the best of them—neither endure nor remain adequate forever. Figure 1 shows by 5-year intervals the accrual of construction needs during the 20 years following the close of the projected 10-year improvement effort. Summation of the 5-year totals reveals that $114 billion in replacement and expansion needs will come due during this 20-year period—a total somewhat greater than the needed investment during the initial 10-year effort.

In figure 2 the needs of Federal-aid highways below the Interstate level are set forth for the primary rural system, for the secondary system, and for Federal-aid highways in urban areas. For the 10-year catch-up period the needs of the primary rural system are estimated at slightly less than $20 billion; those of the Federal-aid secondary system at about $15 billion; and the claims of the Federal-aid urban group are set at nearly $10 billion.

Here even more than in figure 1, the accruing needs after the close of the initial investment period compel attention. During the period 1965–84, Federal-aid primary rural roads (other than Interstate) will require expenditures of $25 billion for replacements and expansion, an amount exceeding the needs of the 10-year accelerated program by 25 percent. The urban routes, requiring as they do relatively large expenditures for the long-lived elements, right-of-way and structures, make relatively modest demands during the 20-year period; but the secondary system requires $21 billion, nearly 40 percent in excess of the outlay during the 10-year catch-up period.

The two sets of bars at the right of the graph give an indication of the accrual of highway needs in the two 5-year periods between 1985 and 1994. Although the values given do not have high standing as a forecast, it can be said that if normal expectancies with respect to needed replacement and needed additions to capacity come to pass, the highway needs during the fourth decade will be something like those shown in the chart. Long-term predictions such as this are useful in financial planning.

Figure 3 (p. 156) shows the needs accrual profiles for roads and streets not eligible for Federal aid, with a similar projection of the forecast to 1994. Here the ratios of replacement and expansion needs to those of the initial program are, relatively speaking, rather high. It is clear that on these lower highway systems the demand for a short-term catch-up program is less urgent than on the systems of greater traffic importance; but the need for a sustained effort over the long pull is plainly evident. This is especially true of the rural group, which includes some State highways, but mostly county and local roads. Replacement and expansion needs are very substantial even in the decade following the 10-year accelerated program. The extended forecast hints that needs in the fourth decade will be even greater than those of the first decade, or catch-up period.[2]

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[1] This article was presented at the 35th Annual Meeting of the Highway Research Board, Washington, D. C., January 1956.

[2] A more thorough discussion of the estimated needs of the several road and street systems, as found in the study made pursuant to Section 13 of the Federal-Aid Highway Act of 1954, is given in the report, *Needs of the highway systems, 1955–84*, H. Doc. 120, 84th Cong., 1st sess.

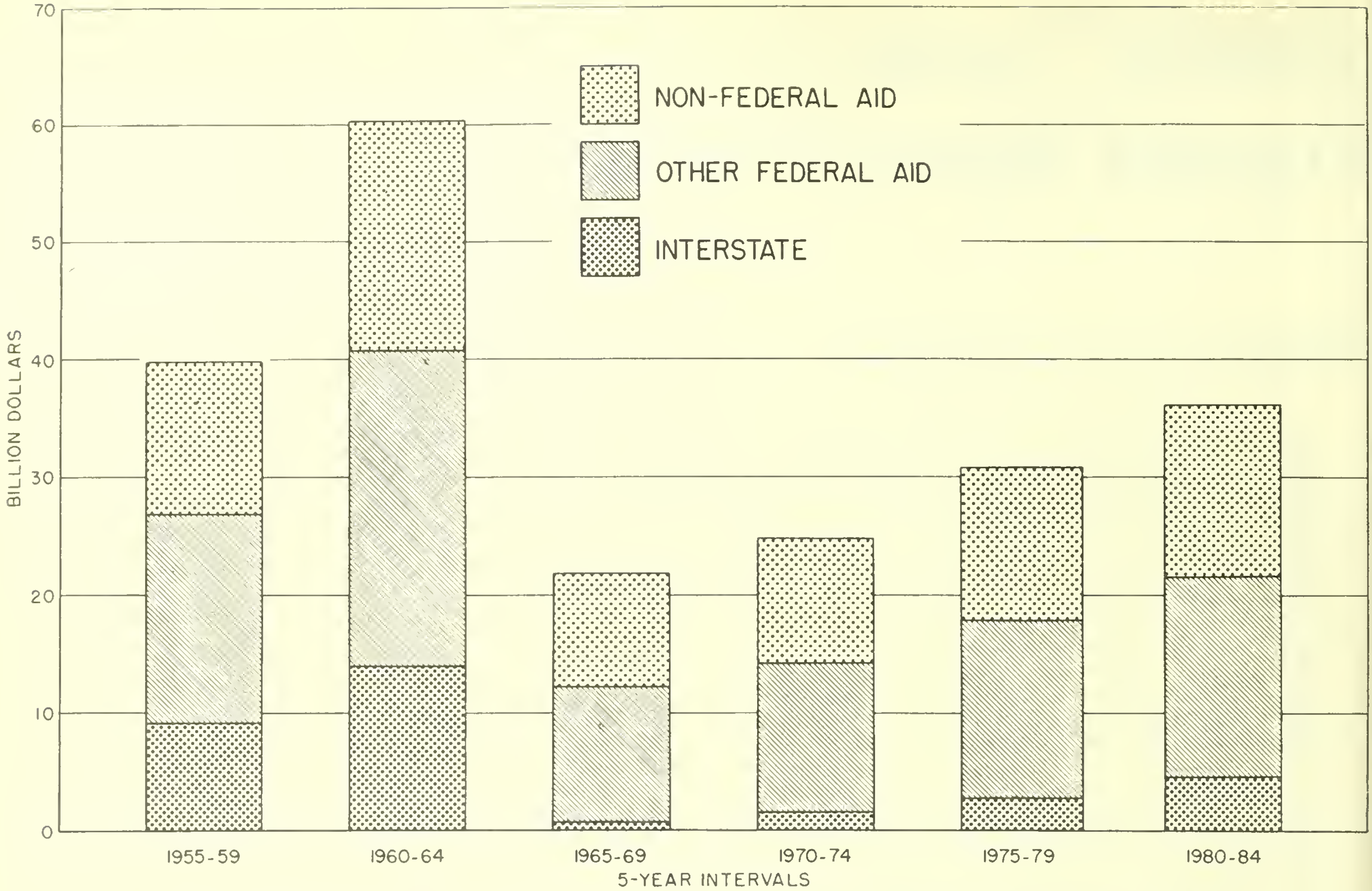

Figure 1.—Estimated highway needs of all roads and streets in the continental United States, 1955–84.

The data shown graphically in figures 1–3 are presented numerically in table 1.

Requirements of Financial Planning

These several needs profiles have been exhibited in order to underline the long-term character of the highway finance problem. In devising an accelerated program to satisfy immediate needs, no State can afford to neglect the accrual of further capital requirements in the years that follow. The task of financial planning, thus complicated by the necessity to peer into the future, is of equal difficulty and stature with that of engineering planning. An unusual combination of technical proficiencies, combining engineering, economics, and statistics, is needed for this work, both by the research and planning staffs of the highway departments and by the experts who may be called in as investigators or consultants.

A successful financial plan must meet four requisites: *First*, to provide for completion of the accelerated highway improvement program within the desired number of years; *second*, to take care of the expenses of maintenance, operation, administration, service of preexisting debt, and other regular commitments of the highway department; *third*, to meet interest and principal charges on the debt, if any, incurred in financing the new program; and *fourth*, to provide sufficient additional revenues to meet the gradually increasing needs for replacement and expansion in the years following completion of the program. The character of the plan best suited to a given situation is largely dependent on the relative magnitudes of the immediate highway needs and those that will accrue in later years. The alternatives range from current-revenue financing to a long-term bond-issue program.

The Cut-and-Fill Concept

A method of analysis designed to produce a plan meeting these requirements has become familiarly known as the cut-and-fill method. In schematic form the concept is illustrated by figure 4. The heavy curved line traces the profile of highway needs: First, those of the initial accelerated or catch-up period, 1956–65; and second, those of the ensuing three decades, during which the needs for replacement and upgrading or expansion of the system develop only gradually. It is evident that a needs profile of this shape lends itself readily to a system of financing whereby a large bond issue sold during the initial construction period can be retired during the ensuing period when replacement and expansion needs are at a minimum.

By inspection the year 1994 was taken as the limit of the bonding term, since the rate of increase of needs accruals begins to fall off at about that time. The heavy straight line represents the rate of revenue supply that will exactly balance needs at the year 1994; it subtends the same area as the profile of highway needs. The area above this line, lying between the years 1956 and 1965, represents bonds issued. The area, equal in size, lying below it and above the needs curve, and spanning the years 1966 to 1994, represents bonds retired. The representation is completed by computing interest at 2 percent in this example—the total revenue requirements of the program being traced by the broken line.

Although the solution is greatly oversimplified in figure 4, it will be observed that, granted the validity of the needs-accrual profile, the financing illustrated on the chart is entirely prudent, since the rate of increase of revenue supply is greater than the rate of increase of accruing needs at the year 1994.[3]

Application of Principle to Federal-Aid-System Needs

Application of the cut-and-fill principle to an actual situation requires a method of

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[3] For somewhat similar treatments of bond-issue financing in relation to the accrual of highway needs, see *Supplemental bond financing for acceleration of the Ohio highway program*, by Bertram H. Lindman, Ohio Department of Highways, 1951: also *Economics of alternative highway programs*, by J. P. Buckley, Automotive Safety Foundation, Washington, D. C., June 1955.

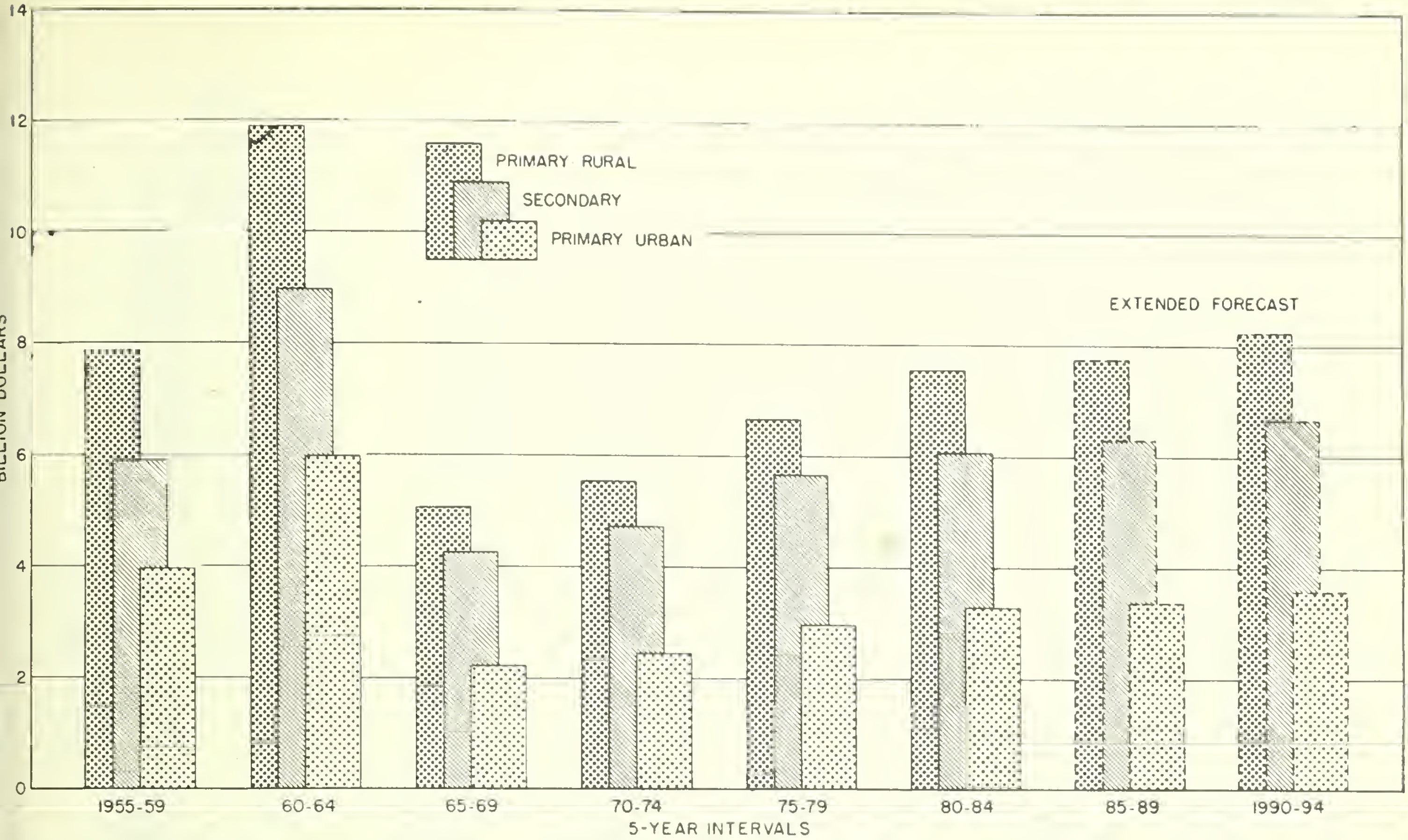

Figure 2.—Needs of the Federal-aid systems other than Interstate, with extended forecast to 1994.

uccessive approximations whereby the existng highway needs, the bond issue, and the equired revenues are brought into a consistent elationship. The major ingredients are a ong-term profile of highway needs and a orecast of available highway revenues at xisting rates over the same period of years, both dependent for their validity upon an adequate forecast of travel volumes. The result is not a decision of policy, but in effect the determination of the maximum prudent bond issue and the minimum prudent increase in the level of highway taxes that will finance the needed improvement program.

To illustrate this method of analysis the needs of the Federal-aid systems (exclusive of the Interstate) in an average State have been taken. The dimensions of the problems are not dissimilar to those of the State highway system in such a State, since there are State highways not on the Federal-aid systems and,

Table 1.—Estimated highway needs [1] of all road and street systems in the continental United States, by 5-year intervals, 1955–94

Road system	Needs of 10-year catch-up period			Replacement and expansion needs following 10-year catch-up period						Grand total
	1955–59	1960–64	10-year total	1965–69	1970–74	1975–79	1980–84	1985–89	1990–94	
Federal-aid systems:	*Billion dollars*	*Billion dollars*	*Billion dollars*	*Billion dollars*	*Billion dollars*	*Billion dollars*	*Billion dollars*	*Billion dollars*	*Billion dollars*	*Billion dollars*
Interstate:										
Rural	5.0	7.5	12.5	0.4	0.9	1.4	2.4	3.4	3.8	24.8
Urban	4.2	6.5	10.7	.3	.7	1.3	2.2	2.9	3.1	21.2
Total	9.2	14.0	23.2	.7	1.6	2.7	4.6	6.3	6.9	46.0
Other Federal-aid highways:										
Primary:										
Rural	7.9	11.8	19.7	5.1	5.5	6.7	7.6	7.7	8.2	60.5
Urban	3.9	6.0	9.9	2.2	2.5	2.9	3.3	3.4	3.6	27.8
Subtotal	11.8	17.8	29.6	7.3	8.0	9.6	10.9	11.1	11.8	88.3
Secondary:										
On State highway systems	3.9	6.1	10.0	2.5	2.9	3.5	3.7	3.8	4.1	30.5
Not on State highway systems	2.0	2.9	4.9	1.7	1.8	2.2	2.4	2.5	2.6	18.1
Subtotal	5.9	9.0	14.9	4.2	4.7	5.7	6.1	6.3	6.7	48.6
Total, other Federal-aid highways	17.7	26.8	44.5	11.5	12.7	15.3	17.0	17.4	18.5	136.9
Total, all Federal-aid systems	26.9	40.8	67.7	12.2	14.3	18.0	21.6	23.7	25.4	182.9
Non-Federal-aid highways:										
Other State:										
Rural	1.5	2.2	3.7	1.1	1.1	1.4	1.6	1.7	1.7	12.3
Urban	.7	1.1	1.8	.4	.4	.4	.5	.5	.6	4.6
Subtotal	2.2	3.3	5.5	1.5	1.5	1.8	2.1	2.2	2.3	16.9
County and other local rural roads	5.3	7.9	13.2	4.3	4.8	5.8	6.5	6.7	7.1	48.4
Local urban streets	5.5	8.4	13.9	3.9	4.3	5.2	6.0	6.1	6.4	45.8
All non-Federal-aid highways:										
Rural	6.8	10.1	16.9	5.4	5.9	7.2	8.1	8.4	8.8	60.7
Urban	6.2	9.5	15.7	4.3	4.7	5.6	6.5	6.6	7.0	50.4
Total	13.0	19.6	32.6	9.7	10.6	12.8	14.6	15.0	15.8	111.1
Grand total, all roads and streets	39.9	60.4	100.3	21.9	24.9	30.8	36.2	38.7	41.2	294.0

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[1] The estimates of highway needs presented in this table are also given, in somewhat more abbreviated form, in the report *Needs of the highway systems, 1955–84*, H. Doc. No. 120, 84th Cong., 1st sess., with the exception that the forecast of future needs is here extended through the year 1994.

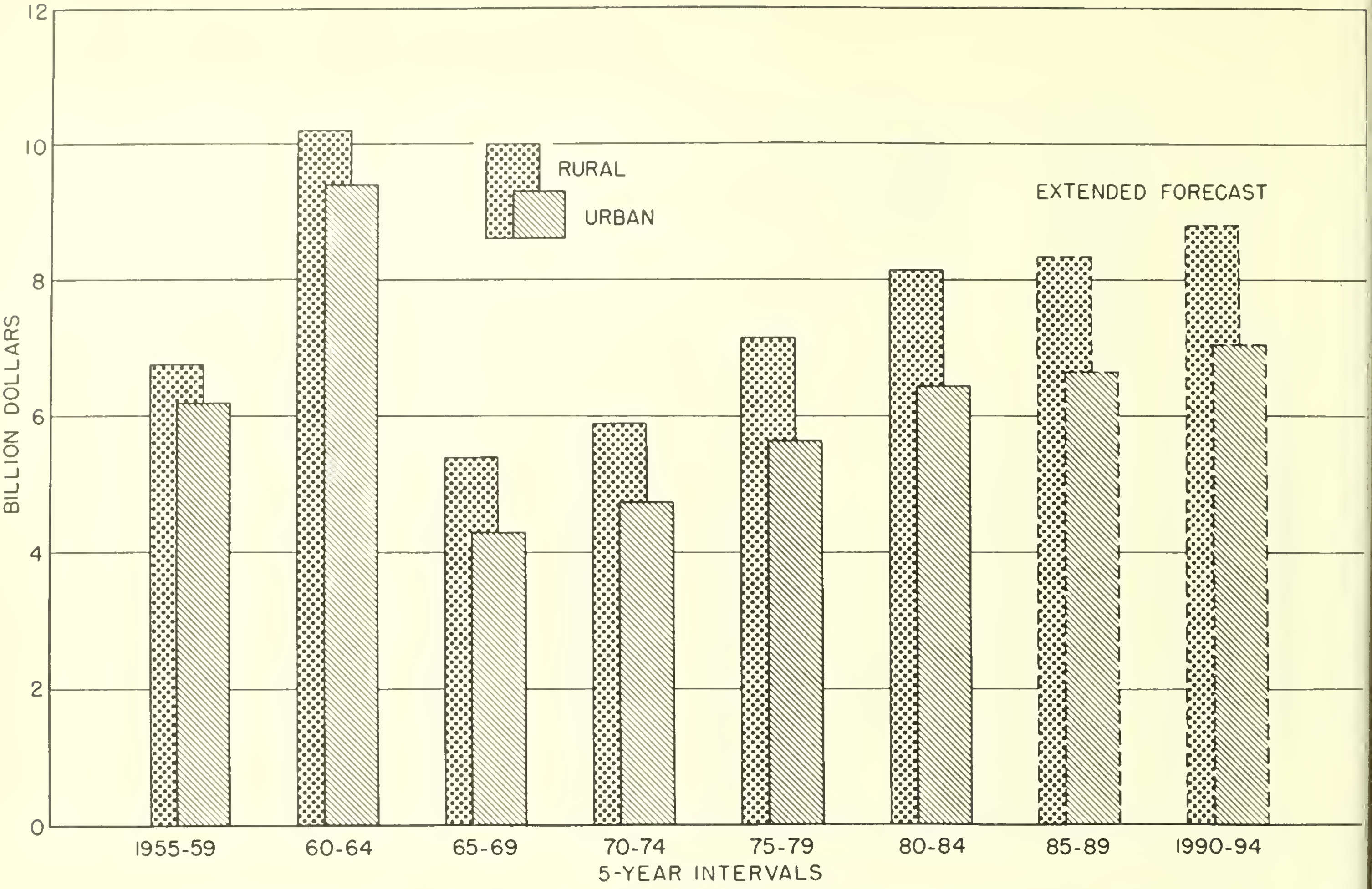

Figure 3.—Needs of road and street systems other than Federal-aid, with extended forecast to 1994.

conversely, Federal-aid secondary highways not on the State highway systems. For reduction to the scale of an average State, nationwide figures, in general, have been divided by 50.

Figure 5 depicts three alternative needs-accrual profile curves for the combined Federal-aid systems (primary rural, primary urban, and secondary) in an average State. Corresponding data are given in table 2. Since the year 1955 is behind us, the time period is taken as 1956–95. The original nationwide highway needs study was based on the assumption of a 10-year catch-up program. In this chart, however, the alternates of a 12-year and a 15-year program are also contemplated. In making the choice between them, a State would have to weigh the advantages of achieving adequacy at an early date against the difficulties in financing, in manpower, and in industrial and organizational capacity of a rapidly stepped-up program.

Because of the probability that a program to be legislated and put into motion in 1956 would have only a minor effect on construction expenditures in that year, the value $48.5 million, predicted on the basis of recent trends, was taken as the 1956 total for all three programs. In each case the expenditures of the catch-up program period have been scheduled so as to rise to a maximum and then recede toward the relatively low level of annual replacement and expansion requirements during the years immediately following the conclusion of the accelerated program.

The differences among the three initial programs are apparent: Average annual expenditures for the 10-, 12-, and 15-year catch-up programs are, respectively, $89, $81, and $73 million. Their cumulative totals, $893, $970, and $1,098 million, respectively, differ because of the accrual of further needs during the longer program periods. Beyond the year 1972 the differences in the accrual of needs are not great. For that reason, and to avoid confusion in the plotting, only the profile of needs for the period following the 12-year program is shown.

Revenues Predicted at Existing Rates

In table 3 and figure 6 we have the second major ingredient of the recipe—highway revenues available to the Federal-aid systems (exclusive of Interstate), as predicted at current tax rates. The left-hand panel shows them classified by source; the right-hand panel by object of expenditure. The predicted revenues rise from $51 million in 1956 to $76 million in 1975 and $100 million in 1995.

By far the largest proportion comes from State revenues—69 percent in 1956, rising to 78 percent in 1975, and 83 percent in 1995. In the year 1954, State government revenues for highways were derived 93 percent from road-user taxes, 4 percent from highway tolls, and 3 percent from general-fund appropri tions and miscellaneous sources. For the calculations user-tax revenues were predict on the basis of the forecasts of travel volun furnished by all States in the 1954 study, wi adjustments for increases in user-tax rat since that time. To avoid a separate predi tion of the future course of toll revenues, ava able to the lower Federal-aid systems chie from toll bridges, funds from this source well as miscellaneous receipts were assum to increase proportionately with user-t revenues.

Increases in local road and street revenu were estimated by reference to recent tren and the predicted trends of general econon indexes.

In order to have an entirely neutral estima of future Federal-aid receipts, it was assum that the current annual authorizations of $3 million for the Federal-aid primary syste $210 million for the Federal-aid seconda system, and $175 million for Federal-aid hig ways in urban areas would be available in t future for the Federal-aid systems outside Interstate. The annual total is $700 milli or $14 million for the average State.

In the right-hand panel of figure 6, exac the same revenues are subdivided by object expenditure. The area shown at the top the chart and diminishing with the ye represents the service of highway debt co tracted prior to the inception of the n

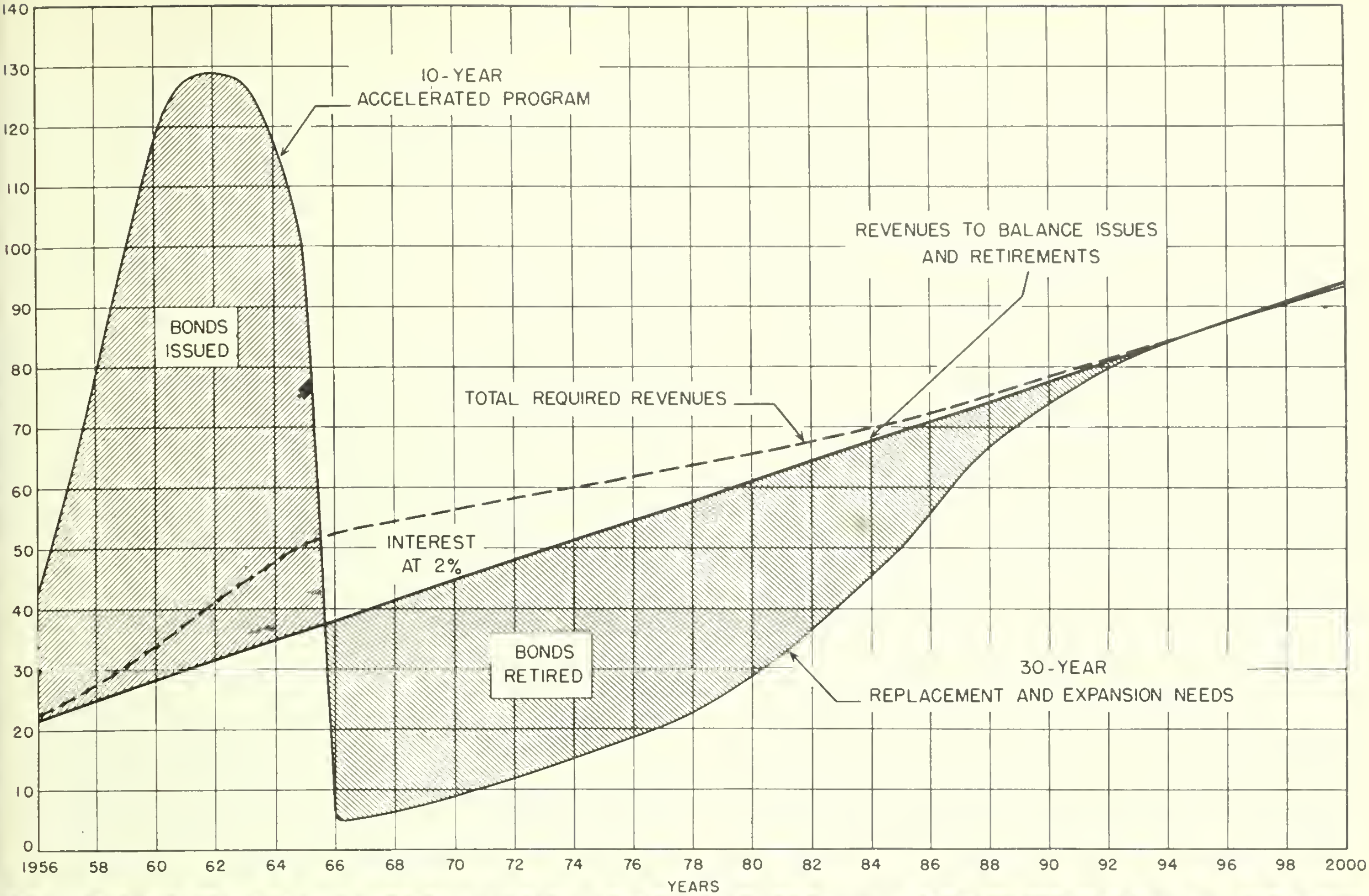

Figure 4.—Illustration of the cut-and-fill concept in financial planning for highways.

program. Beneath this, the predicted expenses of maintenance, operation, and administration are shown, and the net revenues available for capital outlay are given at the base of the chart.

The apparent dip in the trend of funds available for construction during the first few years results from an allowance for special administrative expenses, roughly proportional to capital outlays, during the period of accelerated construction activity. Ignoring this minor variation, it is found that funds available for construction increase from $31 million out of a total of $51 million, or 61 percent, in 1956; to $52 million out of $76 million, or 69 percent, in 1975; and to $74 million (and percent) out of $100 million in 1995. Involved in this trend is the assumption, not unreasonable but of course not inevitable, that the expenses of maintenance, operation, and administration will increase somewhat less rapidly than travel volumes and the revenues derived therefrom, thus gradually releasing a larger proportion for capital outlay.

Needs and Revenues Compared

A comparison of predicted capital needs with predicted revenues available for construction is given in table 4 and figure 7 (p. 160) for each of the three alternate catch-up periods, 10, 12, and 15 years, and for the ensuing decades. In the three upper panels of figure 7, values are expressed in millions of dollars. In the lower panels they are converted to equivalent amounts in cents per gallon of State motor-fuel tax. This procedure takes some liberties with the data, since motor-fuel taxes are only one (although the largest) of the sources from which the revenues of the Federal-aid systems are and will be derived. Equivalent cents per gallon has been found, however, to be the most convenient and most easily visualized unit by means of which predicted revenues may be compared with those required to finance a highway program. In actual practice the required increase in revenues may be distributed among various revenue sources, including increased motor-vehicle imposts, State general funds, highway tolls, local taxes, and Federal funds, as well as the State motor-fuel tax.

The conversion to equivalent cents per gallon was made by the use of a rate of motor-fuel consumption, applicable to all motor vehicles as a group, of 12.73 miles per gallon. On this basis the revenue produced by a tax of 1 cent per gallon is equivalent to about 0.79 mill per mile of travel. Estimates of total vehicle-miles in each year were based on the forecasts made in connection with the 1954 nationwide study of highway needs. No adjustment was made for increased travel volumes and consequent increased revenues resulting from earlier completion of the needed construction under the 10- and 12-year catch-up programs.

Table 2.—Estimated 10-, 12-, and 15-year catch-up programs for the combined Federal-aid systems (exclusive of Interstate) in an average State, with forecasts of needs through 1995

Time period	Estimated construction needs		
	10-year program	12-year program	15-year program
	Million dollars	*Million dollars*	*Million dollars*
1956	48. 5	48. 5	48. 5
1957	60. 4	57. 2	52. 8
1958	76. 1	66. 8	58. 2
1959	91. 9	77. 4	64. 7
1960	106. 7	87. 9	71. 7
1961	112. 7	96. 6	79. 3
1962	112. 8	101. 4	86. 3
1963	107. 9	101. 3	90. 1
1964	97. 6	97. 3	91. 0
1965	78. 2	90. 4	89. 8
1966	------	79. 7	87. 0
1967	------	66. 0	83. 0
1968	------	------	75. 8
1969	------	------	66. 3
1970	------	------	53. 6
1975	56. 6	56. 2	55. 8
1985	68. 9	68. 9	69. 9
1995	77. 4	77. 1	76. 6
Program totals:			
10-year, 1956–65	892. 8	------	------
12-year, 1956–67	------	970. 5	------
15-year, 1956–70	-- -	------	1, 098. 1
10-year, 1966–75	496. 0	------	------
12-year, 1968–75	------	407. 5	------
15-year, 1971–75	------	------	261. 8
All programs, 1976–85	656. 2	653. 4	652. 6
All programs, 1986–95	727. 7	726. 8	725. 0
All programs, 1956–95	2, 772. 7	2, 758. 2	2, 737. 5

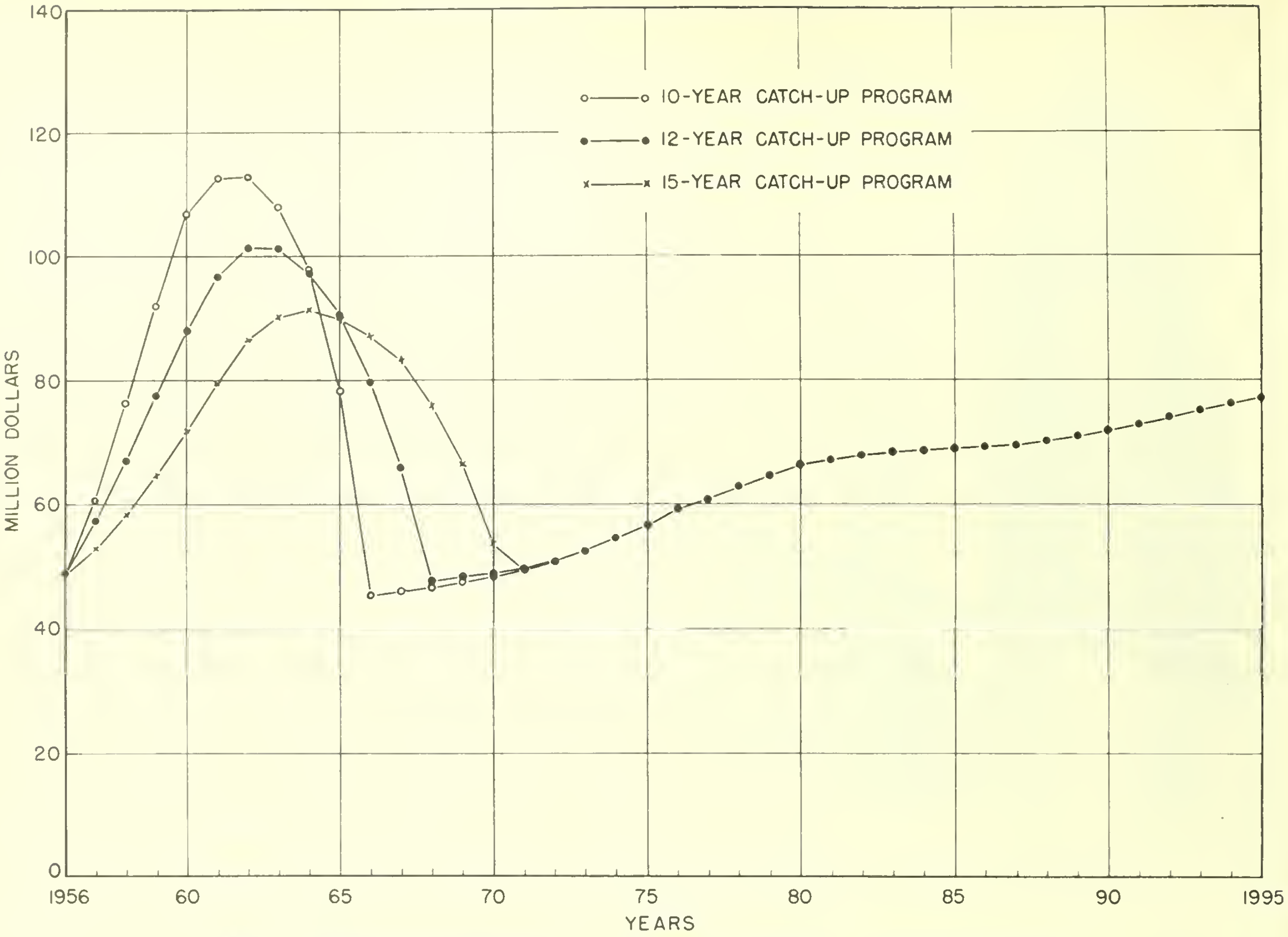

Figure 5.—Needs of the Federal-aid systems in an average State, exclusive of Interstate.

The range in values of annual travel volume in an average State and that of the corresponding yield of 1 cent per gallon State motor-fuel tax are illustrated as follows:

Year	*Annual vehicle-miles of travel*	*Annual yield of motor-fuel tax*
	Millions	*$1,000*
1956	12, 153	9, 547
1965	16, 071	12, 624
1975	20, 258	15, 913
1985	24, 609	19, 332
1995	28, 243	22, 186

The comparison of revenues with needs in figure 7 is presented in pairs of contiguous bars. Heavy dimension arrows indicate the excess of needs over revenues—the additional revenue required under the condition of current revenue financing. In the left-hand panels it is indicated that it would take $893 million or the equivalent of 8.0 cents per gallon of motor-fuel tax to pay the cost of the 10-year catch-up program in this average State. Revenues predicted at current tax rates amount to $344 million, the equivalent of 3.1 cents per gallon. It would take a raise in revenues equivalent to 4.9 cents per gallon to finance this program out of current income. After the close of the 10-year catch-up period, predicted revenues would be very nearly sufficient to meet the accrual of replacement and expansion needs.

A similar story is told in the center and right-hand panels. To meet the needs of the 12-year accelerated program in the years 1956–67 would require additional revenues equivalent to a motor-fuel tax of 4.0 cents per gallon. The 15-year program would require an increase equivalent to 3.0 cents per gallon, making the total requirement nearly double the amount of revenues predicted to be available for capital outlay during the period 1956–70.

This is the picture of current revenue financing, if the goal of producing an adequate highway plant in a reasonable time is to be achieved. It requires a formidable, although perhaps not unthinkable, increase in highway tax rates during the period of accelerated investment. There is a temptation, when confronted with a situation like this, to lower one's sights and decide to "make-do" with something less than adequacy in highway provision and service. Credit financing, however, offers an alternative by which the desired goal may be achieved without putting quite so much strain upon the pocket nerve of the user-taxpayer.

40-Year Bond-Issue Plan

Figure 8 portrays the results of a calculation designed to finance the 12-year catch-up program, plus accruing needs over the following 28 years, by means of a bond issue the total term of which would cover the entire 40-year period. For illustrative purposes the needs-accrual profile has been extended another 5 years. Only capital items relating to the 40-year period are shown, the costs of maintenance, administration, and service of preexisting debt having been deducted at the outset.

Bonds issued, indicated by the mountainous crosshatched area at the left of the chart, amount to $408 million out of a total 12-year investment of $970 million. The equal amount of bonds retired over the ensuing 28 years is shown in similar hatching with reversed slope. The light stippled area represents interest, computed at 2½ percent per year—a rate perhaps somewhere near a median for State issues that may include revenue and limited-obligation bonds as well as those backed by the faith and credit of the State. Total interest payments amount to $243 million over the 40-year period, an average of $6.1 million per year. Interest accounts for only 6.1 percent of the total revenues required for the entire period, or 8.1 percent of the money put into new capital outlay and interest.

The massive area of heavy stipple at the base of the chart represents current revenues applied directly to construction. During the

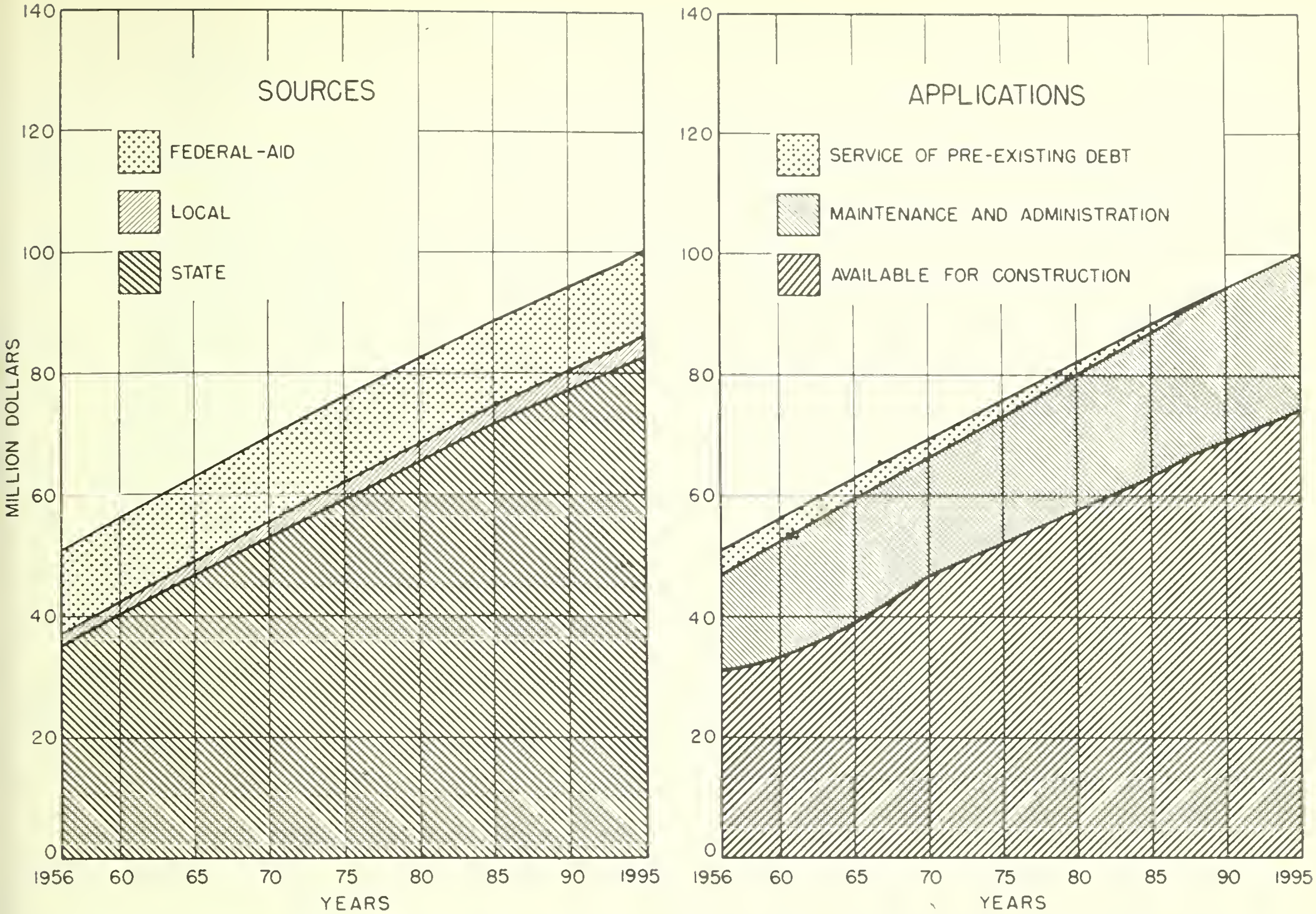

Figure 6.—Predicted revenues of an average State available for the Federal-aid systems, exclusive of Interstate.

nitial 12-year period direct capital outlays of $562 million account for 58 percent of the total capital investment in highways; bond issues for 42 percent. Since all capital outlays in the ensuing 28 years are made out of current revenues, it is clear that the bond issue, although large, plays only a fractional part in the total financial plan.

Table 3.—Predicted revenues of an average State, assuming the continuation of current rates of taxation, available for the combined Federal-aid systems (exclusive of Interstate) for the years 1956–95

Time period	Source of funds				Application of funds (12-year program)		
	Federal aid	State	Local	Total	Service of preexisting debt	Maintenance and administration	Revenue available for capital outlay
	Million dollars	*Million dollars*	*Million dollars*	*Million dollars*	*Million dollars*	*Million dollars*	*Million dollars*
1956	14.0	35.4	1.6	51.0	3.7	15.9	31.4
1957	14.0	36.7	1.6	52.3	4.0	16.6	31.7
1958	14.0	38.0	1.7	53.7	3.9	17.4	32.4
1959	14.0	39.3	1.7	55.0	3.8	18.3	32.9
1960	14.0	40.6	1.8	56.4	3.7	19.2	33.5
1961	14.0	41.9	1.8	57.7	3.7	19.9	34.1
1962	14.0	43.2	1.8	59.0	3.5	20.4	35.1
1963	14.0	44.4	1.9	60.3	3.4	20.7	36.2
1964	14.0	45.7	1.9	61.6	3.3	20.8	37.5
1965	14.0	46.9	2.0	62.9	3.3	20.6	39.0
1966	14.0	48.1	2.1	64.2	3.1	20.4	40.7
1967	14.0	49.3	2.1	65.4	3.0	19.9	42.5
1975	14.0	59.2	2.5	75.7	2.3	21.3	52.1
1985	14.0	72.1	2.9	89.0	1.6	23.9	63.5
1995	14.0	83.0	3.3	100.3	--------	26.0	74.3
Period totals:							
1956–67	168.0	509.5	22.0	699.5	42.4	230.1	427.0
1968–75	112.0	439.0	18.3	569.3	20.9	161.8	386.6
1976–85	140.0	663.1	26.9	830.0	19.1	228.7	582.2
1986–95	140.0	779.7	30.9	950.6	3.0	250.5	697.1
Cumulative total, 1956–95	560.0	2,391.3	98.1	3,049.4	85.4	871.1	2,092.9

Predicted revenues at current tax rates are shown by the heavy continuous line. The total revenues required to finance the program are traced by the heavy stepped line above the stippled area denoting interest. The area between these two revenue lines, denoted by the dimension lines at the right of the chart, represents the revenues required in excess of those predicted. Expressed in equivalent State motor-fuel tax, these additional required revenues amount to 1.415 cents per gallon in each year. Similar calculations made for the 10- and 15-year programs indicate additional revenue requirements equivalent to 1.47 and 1.34 cents per gallon, respectively. If, as seems not unlikely at the time of writing, new Federal-aid legislation should materially increase the authorizations for Federal-aid highways below the Interstate level, the necessity for increased State taxation to finance Federal-aid needs will be correspondingly reduced.

The calculation illustrated in figure 8 serves to demonstrate the manner in which a continuous and gradually increasing flow of revenues may be used to finance a construction program greatly accelerated in a short period of years, to be followed by a relatively moderate buildup of replacement and expan-

Table 4.—Comparison of available revenues with construction needs for the Federal-aid systems (excluding Interstate) in an average State, and the equivalent State motor-fuel tax required to meet those needs

Time period	Estimated construction needs and revenues					Equivalent State motor-fuel tax required to meet construction needs				
	Construction needs	Predicted revenues			Additional required revenues	Construction needs	Predicted revenues			Additional required revenues
		State and local funds	Federal-aid funds	Subtotal			State and local funds	Federal-aid funds	Subtotal	
	Million dollars	*Million dollars*	*Million dollars*	*Million dollars*	*Million dollars*	*Cents/gal.*	*Cents/gal.*	*Cents/gal.*	*Cents/gal.*	*Cents/gal.*
10-year catch-up program:										
1956–65	892. 8	203. 8	140. 0	343. 8	549. 0	8. 035	1. 834	1. 260	3. 094	4. 941
1966–75	496. 0	329. 8	140. 0	469. 8	26. 2	3. 438	2. 286	. 971	3. 257	. 181
1976–85	656. 2	442. 2	140. 0	582. 2	74. 0	3. 687	2. 485	. 787	3. 272	. 415
1986–95	727. 7	557. 1	140. 0	697. 1	30. 6	3. 488	2. 670	. 671	3 341	. 147
Total, 1966–95	1, 879. 9	1, 329. 1	420. 0	1, 749. 1	130. 8	--------	--------	--------	--------	--------
Average, 1966–95	--------	--------	--------	--------	--------	3. 541	2. 504	. 791	3. 295	. 246
Total, 1956–95	2, 772. 7	1, 532 9	560. 0	2, 092. 9	679. 8	--------	--------	--------	--------	--------
Average, 1956–95	--------	--------	--------	--------	--------	4. 319	2. 388	. 872	3. 260	1. 059
12-year catch-up program:										
1956–67	970. 5	259 0	168. 0	427. 0	543. 5	7. 066	1. 885	1. 223	3. 108	3. 958
1968–75	407. 5	274 6	112. 0	386. 6	20. 9	3. 452	2. 326	. 949	3. 275	. 177
1976–85	653. 4	442. 2	140. 0	582. 2	71. 2	3. 672	2. 485	. 787	3. 272	. 400
1986–95	726. 8	557 1	140. 0	697. 1	29. 7	3. 483	2. 670	. 671	3. 341	. 142
Total, 1968–95	1, 787. 7	1, 273. 9	392. 0	1, 665. 9	121. 8	--------	--------	--------	--------	--------
Average, 1968–95	--------	--------	--------	--------	--------	3. 367	2. 400	. 738	3. 138	. 229
Total, 1956–95	2, 758. 2	1, 532 9	560. 0	2, 092. 9	665. 3	--------	--------	--------	--------	--------
Average, 1956–95	--------	--------	--------	--------	--------	4. 296	2. 388	. 872	3. 260	1. 036
15-year catch-up program:										
1956–70	1, 098. 1	354. 0	210 0	564. 0	534. 1	6. 129	1. 976	1. 172	3. 148	2. 981
1971–75	261. 8	179 6	70. 0	249. 6	12. 2	3. 435	2. 357	. 918	3. 275	. 160
1976–85	652. 6	442. 2	140. 0	582. 2	70. 4	3. 667	2. 485	. 787	3. 272	. 395
1986–95	725. 0	557. 1	140. 0	697. 1	27. 9	3. 474	2. 670	. 671	3. 341	. 133
Total, 1971–95	1, 639. 4	1, 178. 9	350. 0	1, 528. 9	110. 5	--------	--------	--------	--------	--------
Average, 1971–95	--------	--------	--------	--------	--------	3. 088	2. 221	. 659	2. 880	. 208
Total, 1956–95	2, 737. 5	1, 532. 9	560. 0	2, 092. 9	644. 6	--------	--------	--------	--------	--------
Average, 1956–95	--------	--------	--------	--------	--------	4 264	2. 388	. 872	3. 260	1. 004

sion needs over a long period. The term of the bond-issue plan (40 years in total, although no bonds would be issued for more than 30) may arouse some objections. The two facts (1) that the financing takes care of all needs for initial construction, replacement, and upgrading during the 40-year period, and (2) that at the close of the period the rate of revenue supply is considerably in excess of the rate of accrual of needs, should be sufficient to quiet such fears. The excess of predicted revenues over predicted requirements may be regarded as a safeguard against unforeseen contingencies.

In a calculation of this sort, the length of

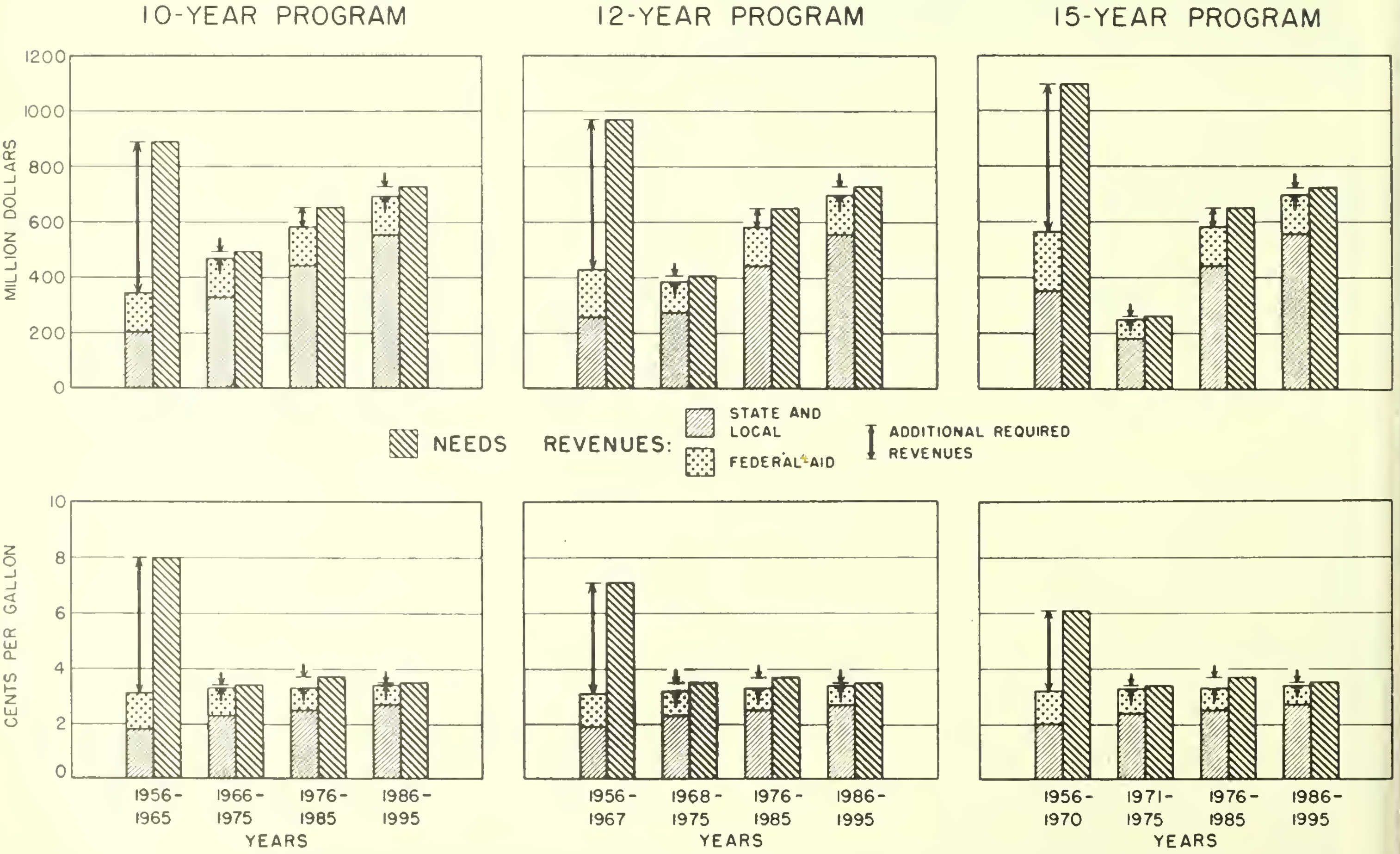

Figure 7.—Comparison of available revenues with construction needs for the Federal-aid systems (excluding Interstate) in an average State, and the equivalent State motor-fuel tax required to meet those needs.

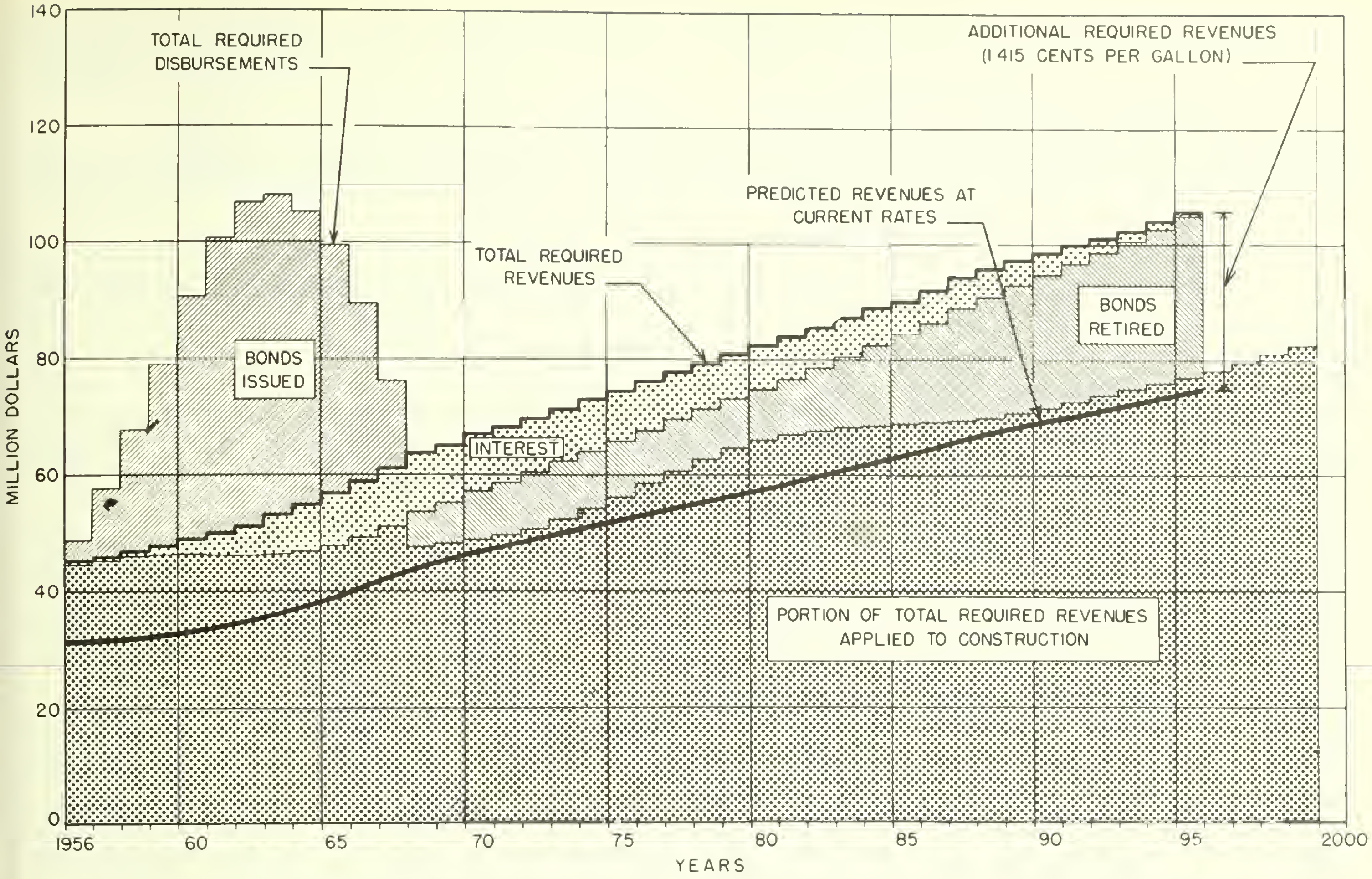

Figure 8.—Calculation of 40-year bond financing plan for the Federal-aid systems, exclusive of Interstate.

the bonding term is contingent upon the composite life span of the highway investment in right-of-way, grading, surfacing, and structures. The results, therefore, tend to set bounds of prudence to (1) term of bond issue, (2) amount of bond issue, and (3) amount or rate of increase in supporting revenues, the limit in the latter case being minimum rather than maximum. In working out the financial plan for an individual State, consideration must be given to other factors, including the general financial situation in the State government, established public policy, and popular attitudes toward credit financing. Alternative plans, with varying terms of bond issue and varying levels of increased revenues, must necessarily be developed before a decision is reached.

Method of Calculation

The procedure in the cut-and-fill method of bond-issue calculation is one of successive approximations. Since the profile of needs and the schedule of predicted revenues are known, it is only necessary to determine the rate of *additional* revenue supply that will accomplish the desired financing in the chosen period of years. The estimated additional revenues can be expressed in terms of cents per vehicle-mile of travel, or as in figure 8, in equivalent cents per gallon of State motor-fuel tax. The first estimate, and the calculation based on it, may be wide of the mark; but repeated estimates will rapidly converge toward a rate of increased revenues that will just pay off the bond issue in the year selected as the final date of retirement.

The process of calculation is illustrated by table 5 which gives values for each year of the entire 40-year period. In order that the variation of the smaller items may be readily traced, values are given to the nearest thousand dollars. Essentially the computation consists of a year-by-year determination of the following quantities: Bonds to be issued or retired in the year; the amount of revenues directly applicable to construction; and the required interest payments.

The following formulas have been found useful in the calculations for the initial program period, or period of bond issuance. They are based on the assumptions that no bonds will be retired during the issuing period and that the bonds for each year are issued at the beginning of the year.

Let N = Highway needs of a given year.
D = Debt outstanding at end of preceding year.
B = Bonds issued in given year.
I = Total interest paid in year.
R = Total revenues available in year.

Then, if receipts and expenditures balance—

$$R+B=N+I$$
$$B=N-R+I$$

Let i = Annual rate of interest.

Then $$I=i(D+B)$$
$$=i(D+N-R+I)$$

Transposing,

$$I(1-i)=i(D+N-R)$$
$$I=\frac{i}{1-i}(D+N-R) \qquad (1)$$

Solving for B,

$$B=N-R+I$$
$$=N-R+\frac{i}{1-i}(D+N-R)$$
$$=\frac{N-R+iD}{1-i} \qquad (2)$$

These formulas may be readily adapted to varying circumstances of bond issuance and retirement.

Conclusion

Some of the alternatives that may be explored in the analysis leading to a financial plan have been outlined. Under conditions such as those depicted, current revenue financing requires drastic tax increases during the initial catch-up period, with only moderate rates in the ensuing decades. A long-term bonding plan may be financed with a relatively small increase in tax rates, sustained throughout the period. Intermediate choices are offered by bond issues of shorter term, requir-

Table 5.—Calculation of a 40-year bond financing plan for combined Federal-aid systems (exclusive of Interstate) in an average State

Year	Required expenditures						Required revenues						Credit financing		
	Program needs and commitments						Amounts predicted at current tax rates								
	Capital outlay	Maintenance and administration	Service of debt outstanding at end of 1955	Total	Interest on new financing at 2½ percent	Total	Federal aid	State revenues	Local rural and urban revenues	Total	Additional required revenues [1]	Total	Bonds issued (Jan. 1)	Bonds retired (Dec. 31)	Bonds outstanding (Dec. 31)
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*
1956	48, 520	15, 849	3, 700	68, 069	92	68, 161	14, 000	35, 380	1, 580	50, 960	13, 509	64, 469	3, 692	--------	3, 692
1957	57, 160	16, 627	3, 980	77, 767	389	78, 156	14, 000	36, 660	1, 620	52, 280	14, 003	66, 283	11, 873	--------	15, 565
1958	66, 770	17, 455	3, 880	88, 105	909	89, 014	14, 000	38, 020	1, 660	53, 680	14, 525	68, 205	20, 809	--------	36, 374
1959	77, 350	18, 353	3, 840	99, 543	1, 688	101, 231	14, 000	39, 340	1, 720	55, 060	15, 022	70, 082	31, 149	--------	67, 523
1960	87, 920	19, 166	3, 740	110, 826	2, 728	113, 554	14, 000	40, 660	1, 760	56, 420	15, 515	71, 935	41, 619	--------	109, 142
1961	96, 560	19, 902	3, 720	120, 182	3, 991	124, 173	14, 000	41, 900	1, 800	57, 700	15, 980	73, 680	50, 493	--------	159, 635
1962	101, 410	20, 423	3, 500	125, 333	5, 371	130, 704	14, 000	43, 160	1, 860	59, 020	16, 462	75, 482	55, 222	--------	214, 857
1963	101, 320	20, 692	3, 400	125, 412	6, 743	132, 155	14, 000	44, 440	1, 900	60, 340	16, 943	77, 283	54, 872	--------	269, 729
1964	97, 340	20, 761	3, 340	121, 441	8, 004	129, 445	14, 000	45, 660	1, 960	61, 620	17, 404	79, 024	50, 421	--------	320, 150
1965	90, 450	20, 620	3, 240	114, 310	9, 070	123, 380	14, 000	46, 880	1, 980	62, 860	17, 863	80, 723	42, 657	--------	362, 807
1966	79, 670	20, 342	3, 120	103, 132	9, 833	112, 965	14, 000	48, 100	2, 040	64, 140	18, 327	82, 467	30, 498	--------	393, 305
1967	65, 990	19, 916	2, 980	88, 886	10, 204	99, 090	14, 000	49, 340	2, 080	65, 420	18, 793	84, 213	14, 877	--------	408, 182
Subtotal, 1956-67	970, 460	230, 106	42, 440	1, 243, 006	59, 022	1, 302, 028	168, 000	509, 540	21, 960	699, 500	194, 346	893, 846	408, 182	--------	-------
1968	47, 600	19, 229	2, 920	69, 749	10, 204	79, 953	14, 000	50, 560	2, 140	66, 700	19, 257	85, 957	-------	6, 004	402, 178
1969	48, 200	19, 512	2, 860	70, 572	10, 054	80, 626	14, 000	51, 800	2, 180	67, 980	19, 724	87, 704	-------	7, 078	395, 100
1970	48, 900	19, 779	2, 620	71, 299	9, 877	81, 176	14, 000	53, 040	2, 220	69, 260	20, 189	89, 449	-------	8, 273	386, 827
1971	49, 700	20, 051	2, 620	72, 371	9, 671	82, 042	14, 000	54, 200	2, 260	70, 460	20, 626	91, 086	-------	9, 044	377, 783
1972	50, 600	20, 307	2, 700	73, 607	9, 445	83, 052	14, 000	55, 460	2, 300	71, 760	21, 088	92, 848	-------	9, 796	367, 987
1973	52, 200	20, 620	2, 480	75, 300	9, 200	84, 500	14, 000	56, 700	2, 360	73, 060	21, 569	94, 629	-------	10, 129	357, 858
1974	54, 100	20, 949	2, 400	77, 449	8, 946	86, 395	14, 000	57, 980	2, 400	74, 380	22, 054	96, 434	-------	10, 039	347, 819
1975	56, 200	21, 287	2, 280	79, 767	8, 695	88, 462	14, 000	59, 220	2, 440	75, 660	22, 517	98, 177	-------	9, 715	338, 104
Subtotal, 1968-75	407, 500	161, 734	20, 880	590, 114	76, 092	666, 206	112, 000	438, 960	18, 300	569, 260	167, 024	736, 284	-------	70, 078	-------
1976	58, 500	21, 615	2, 080	82, 195	8, 453	90, 648	14, 000	60, 500	2, 500	77, 000	23, 002	100, 002	-------	9, 354	328, 750
1977	60, 700	21, 938	2, 000	84, 638	8, 219	92, 857	14, 000	61, 800	2, 540	78, 340	23, 488	101, 828	-------	8, 971	319, 779
1978	62, 900	22, 239	1, 980	87, 119	7, 994	95, 113	14, 000	63, 100	2, 580	79, 680	23, 977	103, 657	-------	8, 544	311, 235
1979	64, 700	22, 541	1, 940	89, 181	7, 781	96, 962	14, 000	64, 420	2, 620	81, 040	24, 464	105, 504	-------	8, 542	302, 693
1980	66, 100	22, 844	2, 000	90, 944	7, 567	98, 511	14, 000	65, 700	2, 660	82, 360	24, 951	107, 311	-------	8, 800	293, 893
1981	67, 000	23, 079	2, 040	92, 119	7, 347	99, 466	14, 000	66, 960	2, 720	83, 680	25, 428	109, 108	-------	9, 642	284, 251
1982	67, 700	23, 304	1, 860	92, 864	7, 106	99, 970	14, 000	68, 240	2, 760	85, 000	25, 902	110, 902	-------	10, 932	273, 319
1983	68, 300	23, 524	1, 840	93, 664	6, 833	100, 497	14, 000	69, 500	2, 800	86, 300	26, 377	112, 677	-------	12, 180	261, 139
1984	68, 600	23, 708	1, 760	94, 068	6, 528	100, 596	14, 000	70, 780	2, 840	87, 620	26, 858	114, 478	-------	13, 882	247, 257
1985	68, 900	23, 870	1, 640	94, 410	6, 181	100, 591	14, 000	72, 100	2, 860	88, 960	27, 355	116, 315	-------	15, 724	231, 533
Subtotal, 1976-85	653, 400	228, 662	19, 140	901, 202	74, 009	975, 211	140, 000	663, 100	26, 880	829, 980	251, 802	1, 081, 782	-------	106, 571	-------
1986	69, 200	24, 054	1, 340	94, 594	5, 788	100, 382	14, 000	72, 940	2, 940	89, 880	27, 660	117, 540	-------	17, 158	214, 375
1987	69, 600	24, 305	500	94, 405	5, 359	99, 764	14, 000	74, 140	2, 960	91, 100	28, 100	119, 200	-------	19, 436	194, 939
1988	70, 100	24, 541	440	95, 081	4, 873	99, 954	14, 000	75, 300	3, 000	92, 300	28, 539	120, 839	-------	20, 885	174, 054
1989	70, 900	24, 750	200	95, 850	4, 351	100, 201	14, 000	76, 360	3, 040	93, 400	28, 934	122, 334	-------	22, 133	151, 921
1990	71, 800	24, 964	180	96, 944	3, 798	100, 742	14, 000	77, 440	3, 060	94, 500	29, 330	123, 830	-------	23, 088	128, 833
1991	72, 900	25, 168	160	98. 228	3. 221	101, 449	14, 000	78, 500	3, 120	95, 620	29, 725	125, 345	-------	23, 896	104, 937
1992	74, 000	25, 371	100	99, 471	2, 624	102, 095	14, 000	79, 560	3, 140	96, 700	30, 120	126, 820	-------	24, 725	80, 212
1993	75, 100	25, 595	-------	100, 695	2, 005	102, 700	14, 000	80, 620	3, 180	97, 800	30, 514	128, 314	-------	25, 614	54, 598
1994	76, 100	25, 794	-------	101, 894	1, 365	103, 259	14, 000	81, 800	3, 240	99, 040	30, 954	129, 994	-------	26, 735	27, 863
1995	77, 100	26, 013	-------	103, 113	697	103, 810	14, 000	82, 980	3, 300	100, 280	31, 393	131, 673	-------	27, 863	-------
Subtotal, 1986-95	726, 800	250, 555	2, 920	980, 275	34, 081	1, 014, 356	140, 000	779, 640	30, 980	950, 620	295, 269	1, 245, 889	-------	231, 533	-------
Grand total	2, 758, 160	871, 057	85, 380	3, 714, 597	243, 204	3, 957, 801	560, 000	2, 391, 240	98, 120	3, 049, 360	908, 441	3, 957, 801	-------	408, 182	-------

[1] Equivalent to 1.415 cents per gallon of State motor-fuel tax.

ing greater revenue increases at the outset but less total cost. The suitability of different solutions depends in large part upon the contour of the needs-accrual profile. If the immediate needs are large in comparison with those of the next two or three decades, a long-term bond issue such as that portrayed in figure 8 is a valid solution. If the immediate needs are only moderate in comparison with those of subsequent years, either current revenue or short-term bond-issue financing is indicated.

The necessity to pay interest makes all bond-issue financing of greater total cost to the State than financing with current funds. That the cost to the taxpayers may be less rather than more is sometimes overlooked. Those who contribute to the support of the highways have alternate uses for their money, yielding either profits or tangible satisfactions, which they must forego in part if increased taxes are paid. The extent of the sacrifice is best measured by the interest-earning power of the money if invested privately, which would generally be more than it would cost the State to borrow the same funds. Thus a bond issue may give highway users and other taxpayers a better bargain than a drastic raise in taxes to finance a current revenue plan.

Applications of Electrical Resistivity Measurements to Subsurface Investigations

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by R. WOODWARD MOORE, Head,
Geophysical Explorations Unit

There has been a constantly growing trend towards the use of the earth-resistivity test on a wide variety of subsurface problems in many parts of the world. In this country the Bureau of Public Roads has been one of the pioneers in establishing the practical application of geophysical test procedures to highway construction problems. Use of the geophysical tests, the earth-resistivity test in particular, has increased rapidly during the past 10 years. At the present time 26 State highway departments have earth-resistivity equipment or have arrangements with other State agencies for cooperative resistivity work. Four States have refraction seismic equipment also. Ten other States and the Territory of Hawaii have indicated keen interest in their possible use of the resistivity test.

The work being done by the Bureau of Public Roads has been expanded during the past 2 years with the construction and purchase of 6 additional sets of resistivity equipment for use by its regional offices in this country, in the Philippines, and in Central America.

The fundamental principles of the earth-resistivity test are reviewed very briefly and a number of graphs are presented which show test data typical of that obtained in various parts of the United States and in Central America, together with correlating drill data. These tests involved such subsurface problems as bridge foundations, slope design for grading projects, landslide conditions and the location of materials of construction such as sand and gravel.

The geophysical tests are not intended to supplant conventional direct explorations by auger borings, the core drill or the test pit, although their elimination is possible in some instances. They are being recommended as valuable additional tools for use in making more complete subsurface surveys at less expense.

PERHAPS the first real attempt to correlate earth-resistivity test data with depths to underlying geologic layers was made by Messrs. Gish and Rooney about 30 years ago. Since that time there has been a constantly growing trend towards use of this relatively simple test procedure on a wide variety of subsurface problems in many parts of the world. Its use in mining exploration will be found in almost every country where mining properties exist. The same is true for areas throughout the world where underground water resources are being sought. Much of the ever-increasing store of literature dealing with the use of earth-resistivity apparatus describes its application to water supply problems. More recently the test procedure has been used in the search for uranium ores.

As is generally known, the Bureau of Public Roads has been one of the pioneers in seeking to establish the practical application of geophysical methods of test to highway construction problems. In this country during the past 10 years there has been a relatively rapid ncrease in the use of the earth-resistivity test, and to a lesser extent, the refraction seismic test on shallow subsurface explorations for highway and other engineering structures. Figure 1 shows the extent to which geophysical tests are being used currently in the 48 States. As of January 1, 1957, 26 State highway departments had acquired earth-resistivity equipment or had arrangements with other State agencies for cooperative resistivity work. Four of these have refraction seismic equipment also. Two other States have made limited use of the geophysical tests in the past. At the present time, 10 additional States and the Territory of Hawaii have indicated an interest in the possibilities of the resistivity test in their subsurface exploration programs.

In furthering the interest in these economical exploration methods, the Bureau has made demonstration tests or conducted full-scale field studies in 33 of the 48 States, 4 of the 7 Central American countries, and in Hawaii. Such work on one project, a 19-mile section of the Baltimore-Washington Parkway in Maryland just north of the boundary of the District of Columbia, made possible a monetary savings on required quantities of sand and gravel sufficient to reimburse the Bureau for all of its expenditures for the geophysical research work since its inception in 1933.

In a further expansion of the work of the Bureau in this field, the Division of Physical Research recently built 6 additional sets of resistivity equipment, 5 of which were fur-

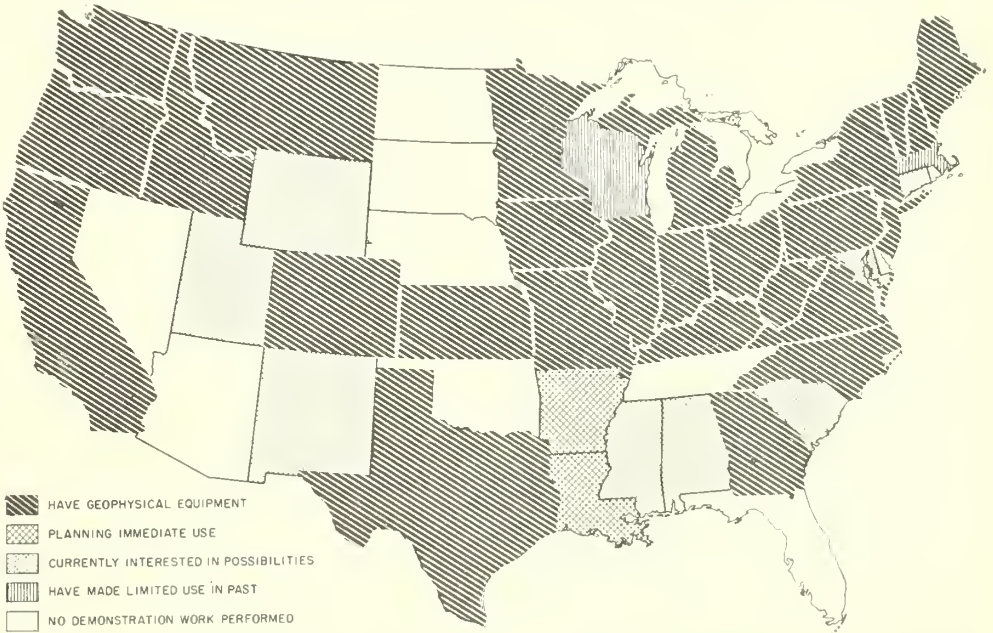
▨ HAVE GEOPHYSICAL EQUIPMENT
▩ PLANNING IMMEDIATE USE
☐ CURRENTLY INTERESTED IN POSSIBILITIES
▥ HAVE MADE LIMITED USE IN PAST
☐ NO DEMONSTRATION WORK PERFORMED

Figure 1.—Map showing current use (January 1957) of geophysical methods in highway construction.

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[1] This article was presented at the 1956 annual meetings of the Southeastern Association of State Highway Officials and the American Association of State Highway Officials at Roanoke, Va., and Atlantic City, N. J., respectively.

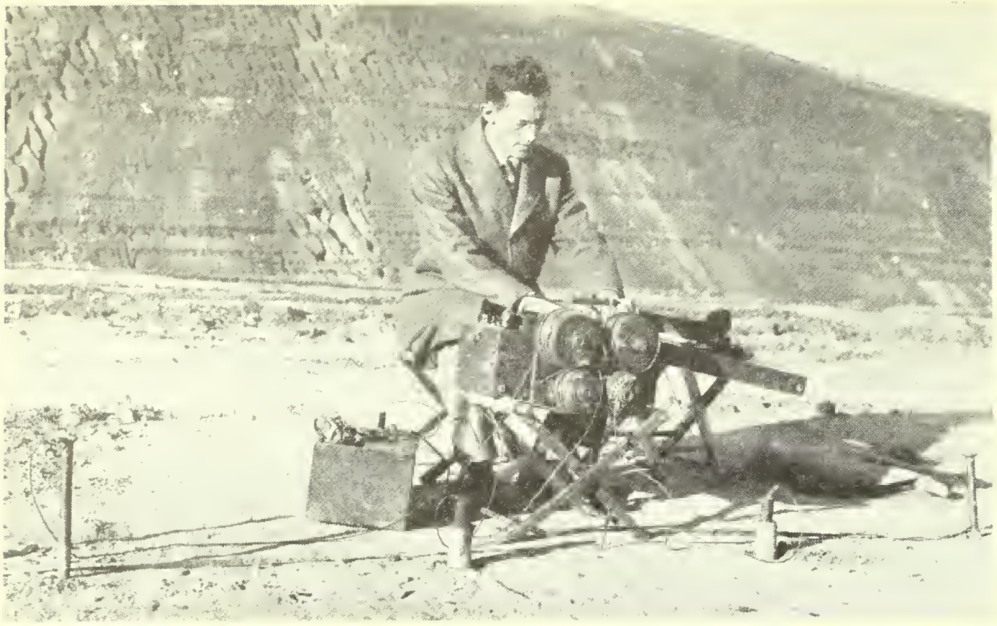

Figure 2.—Earth-resistivity apparatus used by the Bureau of Public Roads in making subsurface explorations.

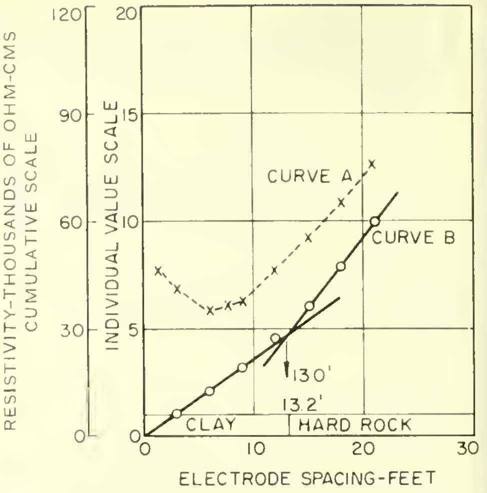

Figure 4.—Empirical method of analysis applied to resistivity curve for a clay stratum with underlying rock in the Washington, D. C., area.

nished to 3 of the Western Regional offices, the Philippine office, and the Inter-American Highway office. One additional, commercially built unit was purchased recently by the Region 15 office for use in its Saint Paul, Minn., District.

With this brief summary it is of interest to consider some of the results obtained thus far. The following discussion is limited to information from the Bureau's files, since a complete record of results obtained in the various States is not available.

Principles Underlying Test

Before presenting examples of the test results, however, a very brief discussion of the principles underlying the test procedure may be of interest. All surface and subsurface materials have, to some degree, the ability to conduct an electrical current. The moisture within the earth's materials and, more particularly, the impurities in this moisture provide an avenue for the current flow. It is natural, then, for the harder more dense rock layers, containing less moisture and perhaps less impurities than the clays and silty layers, to have high resistivity as compared with the resistivities usually found to be associated with the fine-grained clays or silty materials. Salt water is a very good conductor of current flow. Hence a geologic layer such as shale, clay, and so forth, laid down under marine action usually has a very low resistivity. Such materials can be located beneath weathered or leached-out materials overlying them.

Usually, any hard, dense layer of parent rock will have a resistivity which is different from its weathered components. Because of these and other individual characteristics, trial or calibration tests must be made over exposed or otherwise identifiable formations of each material, known to exist in the area of operations, to produce type curves needed in evaluating data obtained elsewhere in the area. Adequate calibration tests are essential to the successful use of the resistivity test. A knowledge of local geology also can be helpful to those making use of the test.

Figure 2 shows the resistivity apparatus used by the Bureau of Public Roads set up in a large cut made to carry the relocated Georgia Railroad around the Altoona Dam which was constructed some years ago. Using the Wenner 4-electrode system, the 4 electrodes are spaced equal distances apart, as shown in the photograph. The distance between the electrodes approximates the depth of soil involved. This is shown to better advantage in figure 3. Hemispherical zones are assumed around each current electrode. Every point on the surface of one of these hemispheres is at the same potential for a given flow of current from C_1 to C_2. The potential electrodes, P_1 and P_2, are placed where the two hemispheres intersect the ground surface, and a measured direct current of a few milliamperes strength is passed through the ground between the two outer electrodes. This produces a drop in potential between the hemispherical zones at C_1 and C_2 which is measured between the two inner electrodes. The resistivity for a centimeter cube of the material is computed and these values are used to plot a resistivity-depth curve such as that shown in figure 4. As previously indicated, the depth of material involved for a 3-foot electrode spacing is approximately 3 feet. By expanding the electrode system to larger separations, the test is carried deeper and any formation such as solid rock, having substantially higher resistivity than the surface layer, will be indicated by a rise in resistivity as shown in figure 4.

The dashed-line relation represents the computed resistivities; that is, actual values obtained in the field test. The circles and the

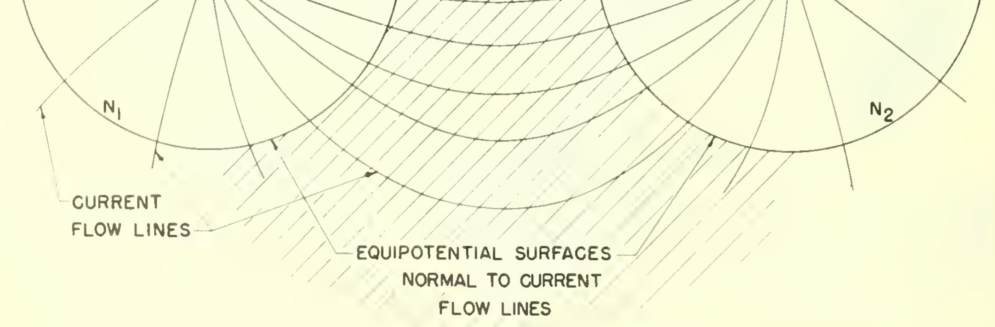

Figure 3.—Equipotential bowls assumed beneath current electrodes when making earth-resistivity tests.

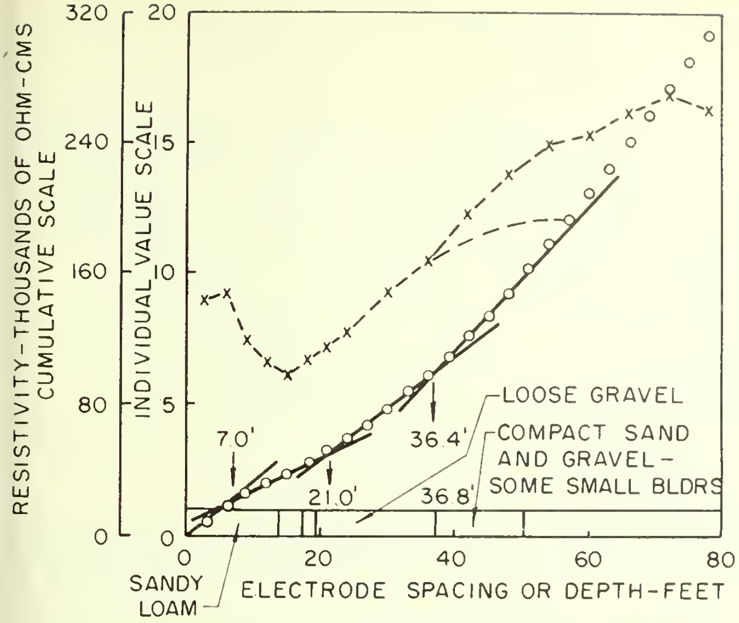

Figure 5.—Resistivity test made at the Fonseca River bridge site in northern Panama.

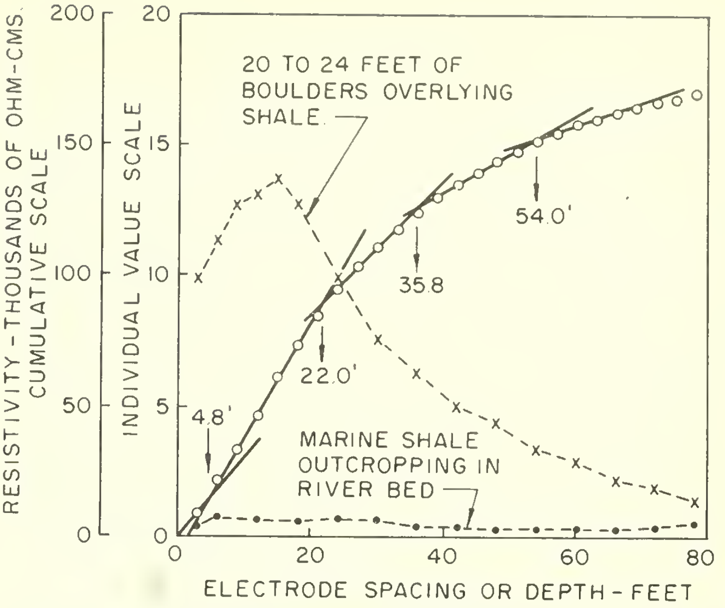

Figure 7.—Resistivity test made at the Corredores River bridge site in southern Costa Rica.

straight-line relations were plotted from computed cumulative values of resistivity. By using a constant increment of electrode spacing of 3 feet, each successive value of resistivity obtained as the electrode separations are increased is added to the summation of all preceding resistivity values to obtain a new point for plotting the cumulative resistivity relation. Drawing the straight lines as shown, their intersection has been found to approximate the depth at which the change in the underlying formation occurs. This empirical procedure, developed by the Bureau of Public Roads, has been used with considerable success in interpreting resistivity data obtained in the field and predicting depths to subsurface geologic formations of interest to the highway engineer.

Much has been written about the analysis of earth-resistivity data. Many of the published reports deal with theoretical procedures based on ideal surface and subsurface layers that are seldom found in field studies. For the convenience of those wishing to pursue the subject further, a brief reference list is given on page 169.

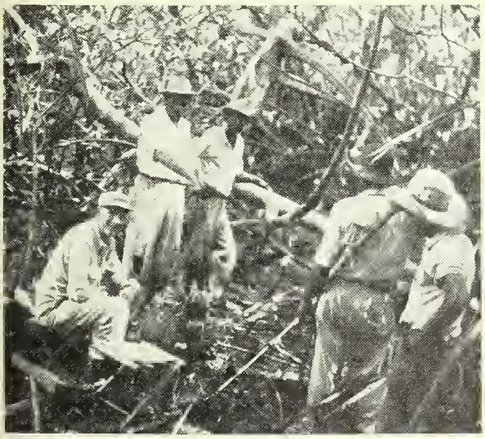

Figure 6.—Surface conditions existing in a mangrove swamp in northern Panama.

Without further consideration of the theoretical aspects, the following paragraphs discuss the results of earth-resistivity tests made in widely scattered areas but selected as typical of the many individual tests involved.

Tests Made on Inter-American Highway

Panama

Resistivity tests made at a bridge site on the Inter-American Highway in northern Panama produced data typified by those shown in figure 5. This test not only located sand, gravel, and boulders where it had been found by a boring but it also showed a change to the more compact material found 15.4 feet below by the boring. The curving dashed line breaking off to the right at an electrode spacing of 36.0 feet shows the shape the field curve would have assumed had the more compact layer not been present. Elsewhere in this area it was necessary to test for another bridge site on an alternate route. Figure 6 shows the almost impossible surface conditions existing in a mangrove swamp. Yet, a 30-minute test located a change to a higher resistivity material 34.0 feet below the surface. It was recommended that a sounding rod be used to confirm the indications of the test. The rod penetrated to within about 1 foot of the depth which the test had indicated. This was a case where the only two practicable methods of test were used, the sounding rod and the portable resistivity apparatus.

Costa Rica

In the jungles of southern Costa Rica nine tests were made in less than a day to explore the foundation conditions for the possible site of a large bridge over the Corredores River. Figure 7 shows data for 2 of the 9 tests. The upper curve was obtained over a 20- to 24-foot-thick boulder deposit resting on shale of marine origin. The lower curve represents data obtained for calibration purposes directly on an exposure of the shale bed. The strong downtrend obtained in all of the curves was interpreted as indicative of the influence of the underlying low resistivity shale bed. Figure 8 shows the calibration test in progress with the boulder bed in the background. It should be emphasized again that trial or calibration tests made over exposed materials, rock, shale, soil, and so forth, are essential as criteria for properly evaluating resistivity data in each area where a survey is to be made. At this bridge site a single drill hole made on the most accessible bank, when considered in conjunction with the resistivity data, served to prove the entire river crossing. A final site selected for the bridge was some distance downstream from this location at a point where the river channel was narrower. However, much costly and unnecessary drilling was eliminated by the resistivity survey made at this location.

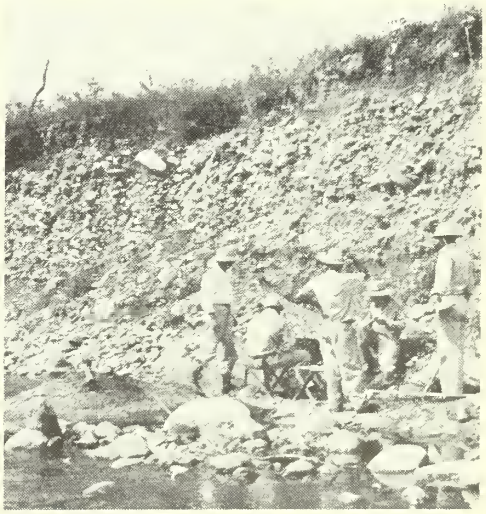

Figure 8.—Resistivity calibration test over marine shale underlying gravel-boulder bed along the Corredores River in southern Costa Rica.

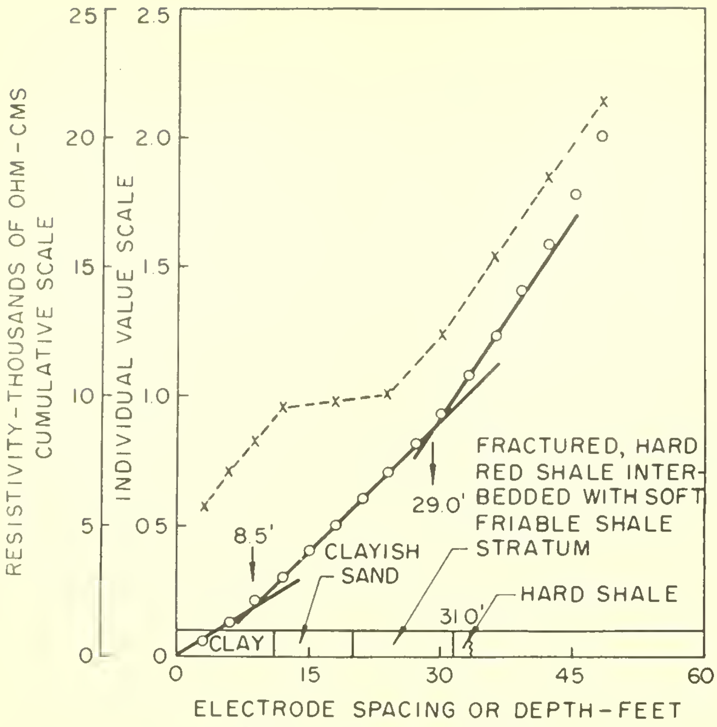

Figure 9.—Resistivity test made at the Limon River bridge site in Nicaragua.

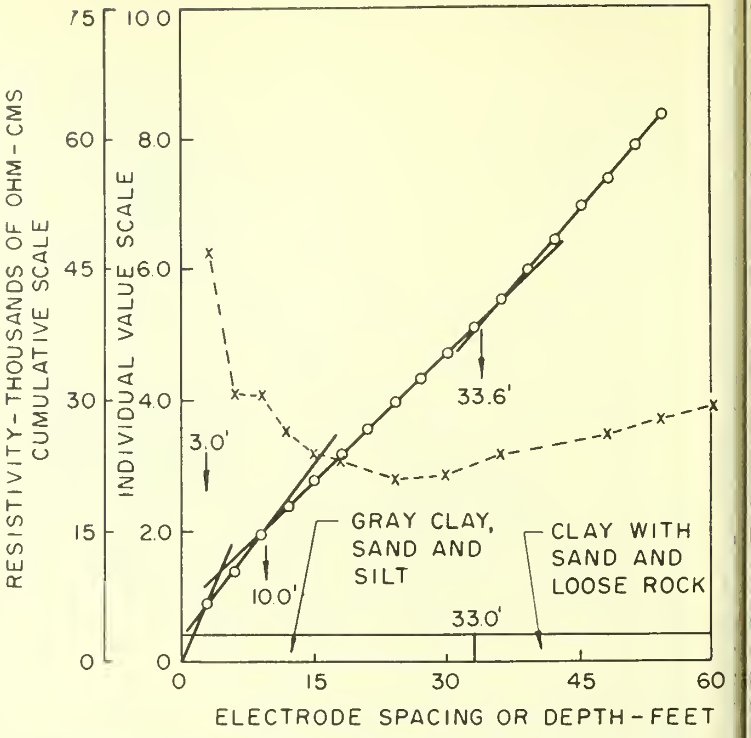

Figure 10.—Resistivity test made at the Rosario River bridge site in Guatemala.

Nicaragua

In Nicaragua the resistivity test was used successfully at the crossing of the Limon River, as shown by the data in figure 9. The strong, definite increase in resistivity at a depth of 29.0 feet was conclusive evidence of a major change in the underlying formation. Since salt-laden materials were involved, the interpretation was that the sharp rise in resistivity resulted from the influence of a more compact, impervious stratum such as the hard shale which had been located at substantially the same elevation by a test boring.

Guatemala

Figure 10 shows data from a test made near Champerico on the Pacific coast of Guatemala. The rise in resistivity indicates a change at a depth comparable to that at which the drill encountered a significant change in the substrata. Here again, since the materials involved were saturated with salt water, the increased resistivity can be interpreted as the influence of a more dense, less permeable stratum likely to have better foundation characteristics.

Slope Design Investigations

In slope design it is essential to know the character of the materials present in the proposed cuts. Lower unit bid prices for unclassified excavation result when earth slopes predominate in the plans and specifications. Accurate information regarding the materials will tend to prevent claims for underrun or overrun items. In a recent case, a bill of nearly $19,000 was paid a contractor as compensation for anticipated profits because final estimate rock quantities underran the contract item by more than 25 percent.

North Carolina

Figure 11 shows resistivity data from a test on a slope design investigation in the western part of North Carolina. The rising trend at a depth of 37.6 feet was indicative of a solid granite bedrock, the lower resistivity, although 150 times as high as a solid rock tested in Kansas, represented a weathered layer easily removed by rooter and self-loading scraper, which required a 1-to-1 slope. Such a test can be made in a matter of 12 to 15 minutes. When such marked differences exist between the resistivities of hard and soft materials in a given area, and they often do, the operator can predict the presence or absence of solid rock at a particular depth within 3 or 4 minutes after completing the test.

Idaho

Landslides plague the highway engineer in many areas. Disturbing Nature's slopes often can be disastrous. The earth-resistivity test can be of considerable help in preliminary explorations to locate possible sliding surfaces or to delineate existing failure zones. Figure 12 shows 3 of 6 sections prepared from investigations made on 6 separate slide areas along the Priest River in northern Idaho. The work was done in 2 days. While the hardpan and less pervious layers inferred from the

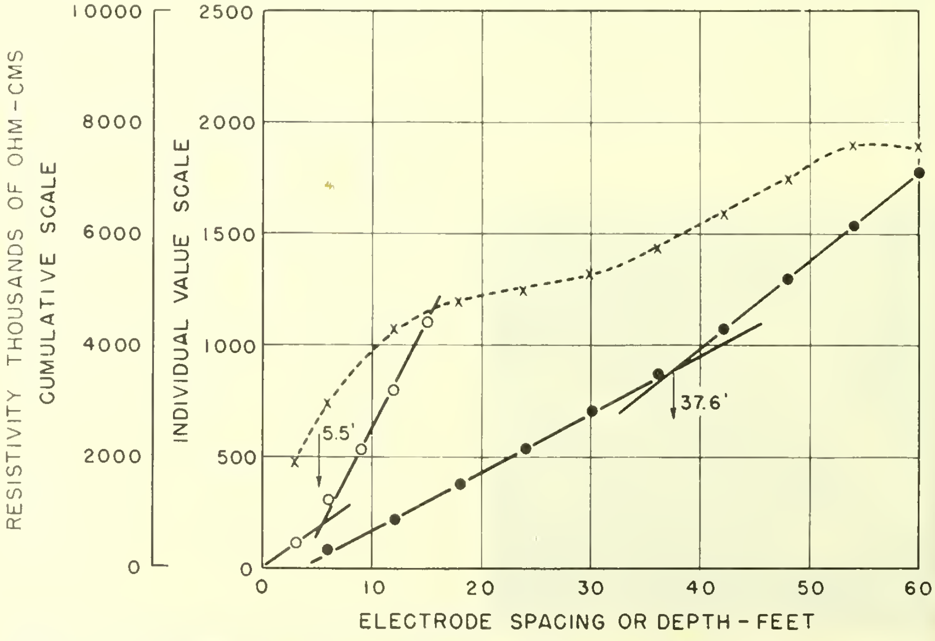

Figure 11.—Resistivity test made on the Blue Ridge Parkway in North Carolina. Test discloses presence of granite rock beneath weathered granite.

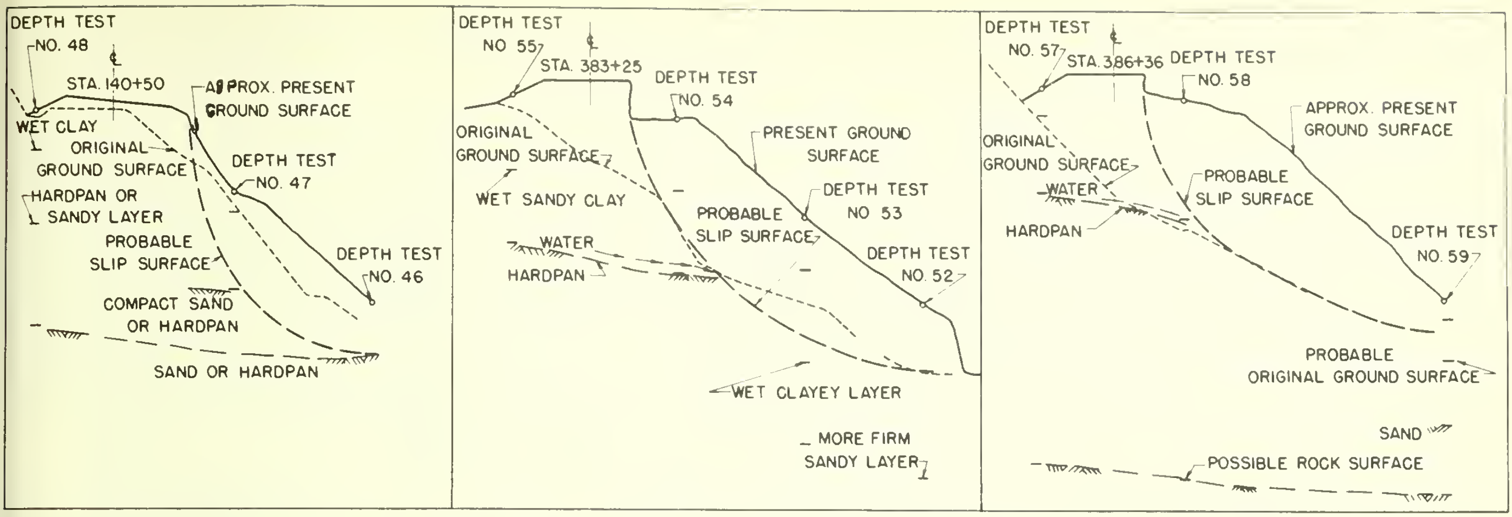

Figure 12.—Sections plotted from earth-resistivity data obtained at three locations along the Priest River in northern Idaho, showing probable slip surface upon which fill material is moving.

resistivity data should not be accepted until confirmed by boring or test pit at one or more points, the preliminary information shown should be of considerable value in locating sites for direct exploration and in laying out drainage operations so as to achieve maximum interception of water percolating through to saturate the fill material and initiate the failures as indicated.

Locating Sand and Gravel Deposits

Mention was made of the successful use of the test procedure in locating sand and gravel. By keeping the electrode system at a constant spacing, say 30 feet or any depth critical to the problem under study, it is possible to make a rapid survey of large areas to locate buried rock or lenses of sand, gravel, special clays, and so forth. Such materials may be in demand for local construction work. Figure 13 shows data from two such constant-depth resistivity traverses made over a deposit of sand and gravel (upper curve) and over a clay formation (lower curve). In this test procedure the electrodes are shifted as a group for successive readings of resistivity as the test progresses along a given line. Any significant change in the materials within the 30-foot depth below the surface is usually indicated by corresponding changes in the measured resistivity values. Any material surrounded and covered by a material of substantially different resistivity characteristics can be located and outlined with regard to its lateral limits by this type of test.

Figure 14 shows a resistivity contour map prepared from data obtained from several constant-depth traverses. The area involved contains large amounts of granular material suitable for stabilizing plastic clay soils before placing a pavement. A comparatively few borings served to prove the existence and identify the character of the granular material in the formation. This material was found within the right-of-way limits near the middle of a section of the Baltimore-Washington Parkway on which large quantities of granular material were needed. Although this particular deposit of granular material was not excavated due to the retention of this area for esthetic and other reasons, large savings in hauling costs in excess of $100,000 were possible elsewhere in the 19-mile project.

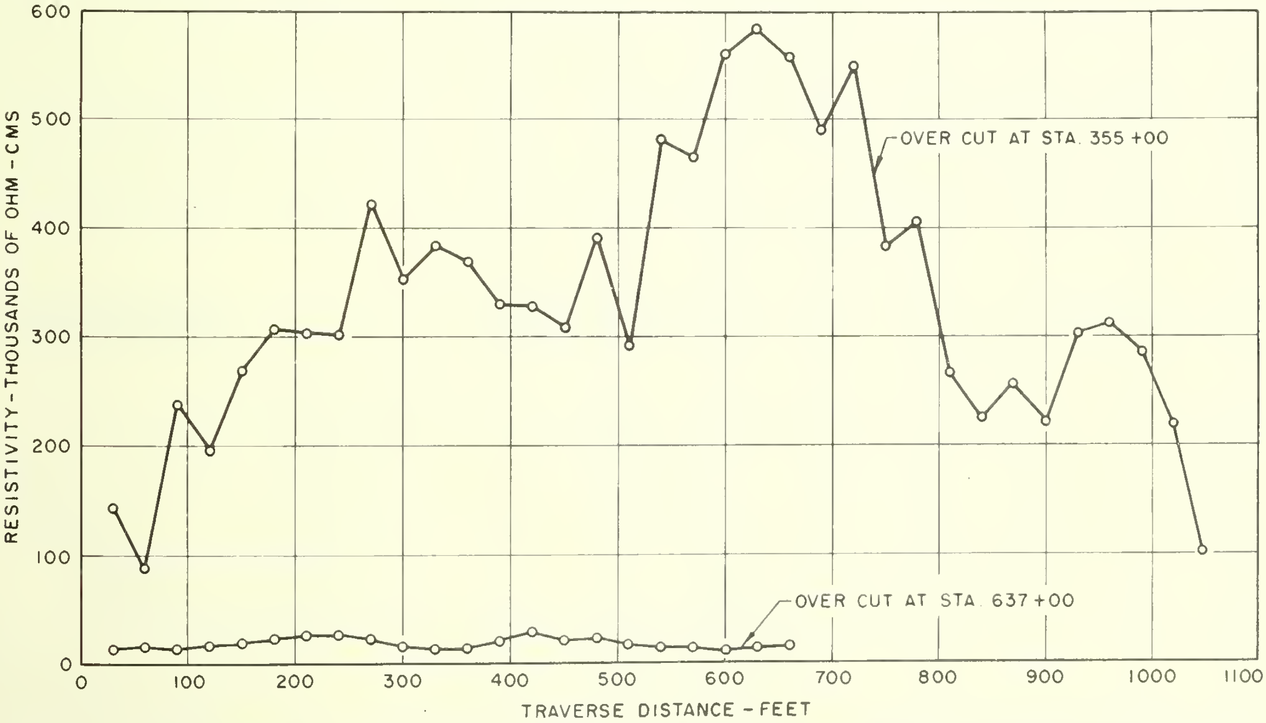

Figure 13.—Constant-depth resistivity traverses over a sand and gravel deposit (upper curve) and over a clay formation (lower curve) in the area of the Baltimore-Washington Parkway. Electrode spacing or depth was 30 feet.

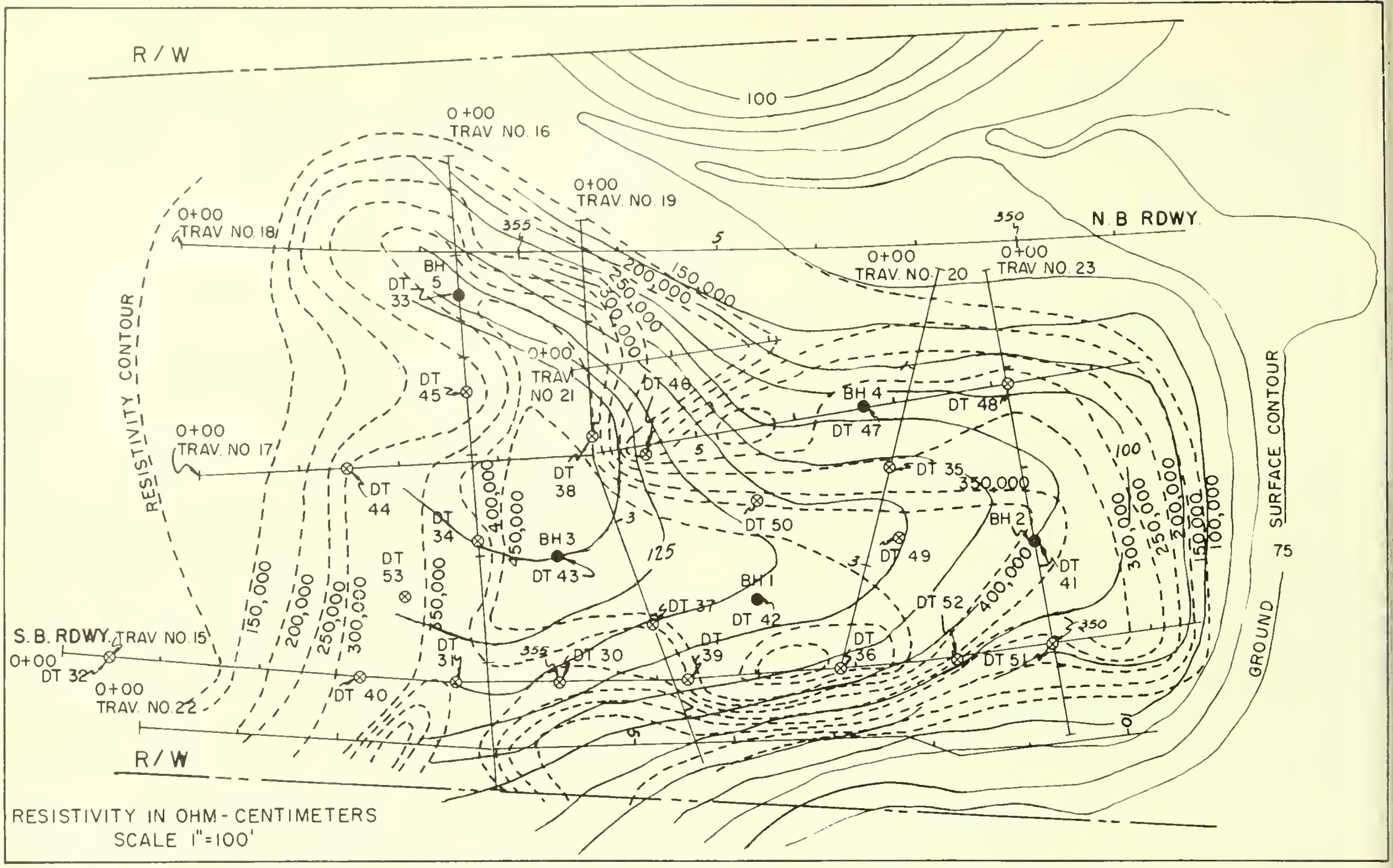

Figure 14.—Resistivity contour map showing sand and gravel deposit within the right-of-way limits of a section of the Baltimore-Washington Parkway.

Benefits of Resistivity Tests

Figure 15 contains data which emphasize the practical application of the resistivity test to three differing subsurface problems. In the left-hand graph the resistivity test has produced a reliable indication of ledge rock at a depth where solid material was encountered by a sounding rod. The sounding rod can be stopped by a boulder or a thin layer of hard material and its use alone may lead to erroneous conclusions. If used in conjunction with the resistivity test, this cannot happen. The strong uptrend in the resistivity curve is conclusive evidence of a definite change in the substrata. The continuing trend upward indicates a thickness of at least 20.0 feet for the underlying formation.

In the middle graph, the resistivity test showed evidence of a low resistivity layer about 3 feet below the deepest boring that had been made in foundation investigations for the structure. This material could be a plastic clay formation likely to allow serious settlement or it could be a fairly dense marine clay with good support for piling. A resistivity survey prior to the boring tests could have dictated the depths to which borings should be carried to obtain samples of the low resistivity layer from which its foundation characteristics might have been established.

Again, in the third graph, good foundation material, waste slag, found near the surface

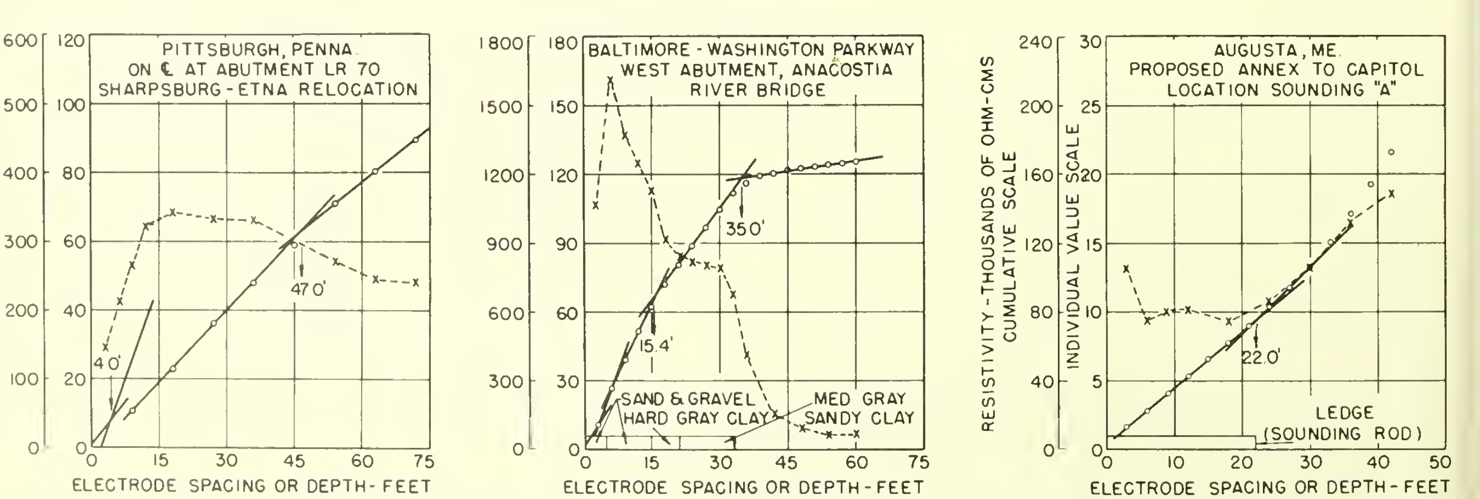

Figure 15.—Earth-resistivity tests applied to three different subsurface problems: corroborating sounding rod data (left), determining proper depth of borings (center), and guaranteeing surface conditions to considerable depths (right).

could be guaranteed to depths sufficiently great to provide ample support for almost any type of structure. The resistivity test ndicated that the old slag dump present at this test location had a thickness of at least 40 feet. At a location only a few hundred feet away, at a proposed pier location, this material was only 13 feet thick. Thus we see that the resistivity test can provide good preliminary information and, working with the sounding rod, drill or auger, also be nstrumental in obtaining more complete information regarding the subsurface materials at a given structure site.

Many more instances could be cited in which the earth-resistivity test has produced very useful results. Although it is not applicable to all combinations of surface and subsurface materials, its successful applications will far outnumber its unsuccessful applications.[2] Often, savings on a single construction project resulting from a thorough resistivity survey are sufficient to pay for the initial cost of suitable equipment as well as several years of fieldwork. The Turkish Department of Highways effected an estimated savings of $200,000 on its first application of the test to the location of sand and gravel. In the work of the Bureau, similar savings have been made possible on the same field problem. As the various agencies now using the earth-resistivity test in this country publish accounts of their experience, other worthwhile savings will no doubt be reported. With a wider and more complete report of the results obtained by those making use of the test, the earth-resistivity test procedure will become more fully established and its possibilities more fully realized by engineers concerned with a study of the subsurface materials.

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[2] There have been a few instances where the resistivity test was inconclusive. Conditions in which the surface layer and the underlying layer have little or no contrast in resistivity have nullified test results. Metal structures such as pipe lines, electric conduits, railroad rails, etc., can cause current to leave the normal path and produce unsatisfactory test results.

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A Study of Factors Related to Urban Travel

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH
BUREAU OF PUBLIC ROADS

Reported by WILLIAM L. MERTZ, Highway Transport Researc
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This study was undertaken to determine the effect of automobile ownership, population density, distance from the central business district, and income per household on the number of vehicular trips residents made in Washington, D. C., on an average weekday in 1948.

It was found that the use of all four variables combined did not produce a significant increase in the accuracy of predicting trips over that which was obtained using automobile ownership and population density combined. Furthermore, automobile ownership was found to be the most reliable single predictor with very little additional accuracy gained by combining it with population density.

THERE are many factors that affect the number of daily trips made by residents of an urban area. It is extremely difficult to find objective measures of any phenomenon that involve the psychology of a large group of people but some inferences may be drawn. It seems quite logical to assume that the number of automobiles owned per dwelling unit by the residents of an urban area is closely related to the number of trips made by the residents of that area. Many more possible factors come to mind. What for instance is the effect of net density of population, distance from the central business district, and income per household on the number of daily trips? There are other factors, too, that may have a bearing on the question.

This report presents the results of an analysis of the effect of automobile ownership, distance from the central business district, income per household, and net population density (population per net residential acre) on the number and frequency of vehicular trips made by residents by all modes of travel.[1] These factors were selected because it has been possible to establish measures that are free from personal bias and because it seems that they constitute a quite logical premise on which to begin an analysis of total number of trips made by the residents of urban areas.

Sources of Information

This analysis was based on travel data obtained from the Washington, D. C., metropolitan area traffic study made in 1948, but was limited to the District of Columbia, because reliable data on income and population density were not readily available for the rest of the Washington metropolitan area.

The income information was obtained from a study made by the Washington Board of Trade.[2] The average gross income per household was estimated for each census tract on the basis of rentals and home values reported in the 1950 Census of Housing. This method, although not a direct measure of income, was the best available information on this subject.

Population data were obtained from the 1948 metropolitan area traffic study and converted to population per net residential acre on the basis of factors used by the Department of Research and Statistics, United Community Services, Washington, D. C. The residential acreage was based on the July 1948 areas used for residences. All other land uses such as streets, parks, commercial and industrial areas were excluded. Land occupied by the United States Soldiers Home and St. Elizabeths Hospital was also excluded.

Data relative to automobile ownership and distance from the central business district were obtained from the 1948 Washington, D. C., metropolitan area transportation survey. Census tracts were utilized for this study since population density and income data were reported only by those areas. The 1948 transportation trip data, though summarized by census tracts, did not separate trips by the various modes of travel, thus preventing in this study a separate analysis o either of the two primary vehicular modes o travel.

Correlation of Trips and Automobil Ownership

For each census tract the average numbe of residents' trips per dwelling was correlate with the average number of automobile owned per dwelling. The data are presente in figure 1. A very definite relation is indicated but since the points are somewha scattered, the principle of least squares wa used to fit a line through the plotted point of the trip averages. This is known as a regression line. The equation of this lin is $X_1=2.88+4.60X_2$ where X_1 is residents' trips per dwelling unit and X_2 is automobil ownership per dwelling unit. The correlatio coefficient is +0.827.

Since correlation coefficients range from 0.00 when there is no correlation betwee two variables to ±1.00 for perfect correlation, they serve as a measure of the strength of association between the variables. For a perfect correlation, every plotted point would fall exactly on the regression line, and for a condition with no correlation the scatter of points would have a round pattern with the highest density at the center. The resulting regression line would be horizontal showing no change of the dependent variable on th

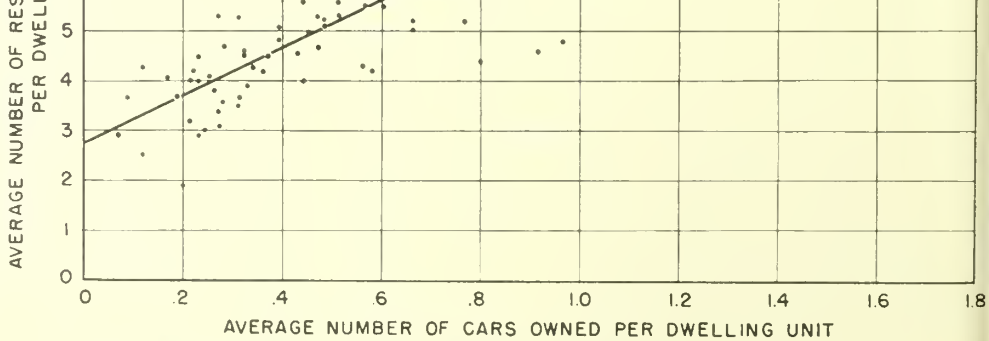

Figure 1.—Relation of resident vehicular trips to automobile ownership.

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[1] All modes of travel refers to residents' trips within the metropolitan area by various forms of transportation. It does not include trips by taxi and truck operators in the course of their daily work.

[2] *Number of households and average income per household in the District of Columbia, 1949–1954,* by the Economic Development Department, Washington Board of Trade, June 1954.

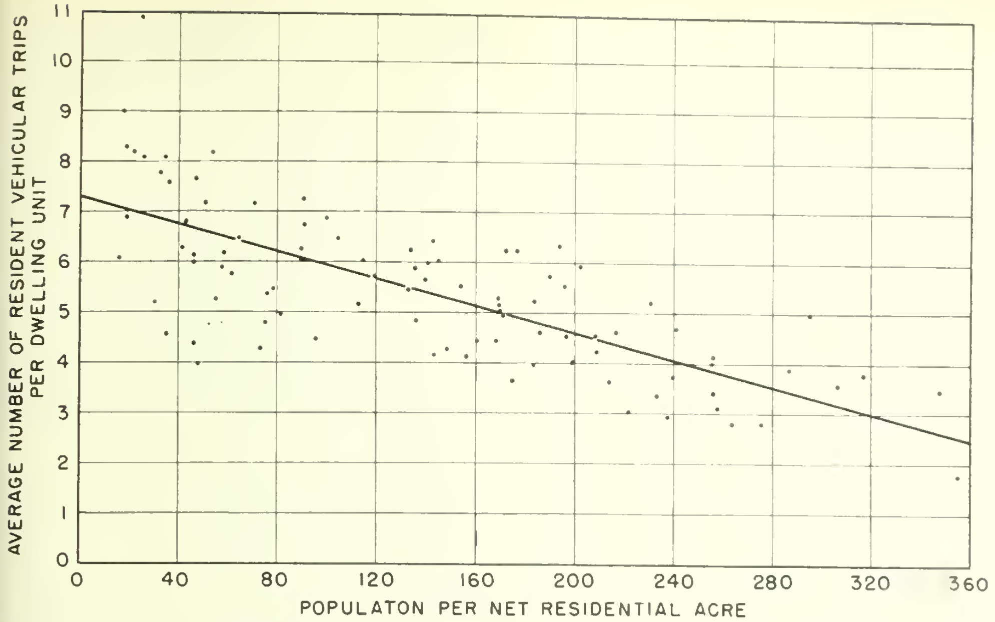

Figure 2.—Relation of resident vehicular trips to population density.

vertical axis with a change in the independent variable on the horizontal axis. Another interpretation of the coefficient of correlation is that its square multiplied by 100 is the percentage of the total variation in trips that can be explained by variation in automobile ownership or any other factor being studied, such as income, population density, or distance. Conversely, the quantity 1 minus the square multiplied by 100 is the percentage of variation left unexplained or due to other causes.

The square root of the average of the variation left unexplained is the standard error of estimate and for the data plotted in figure 1 (relation of resident trips to car ownership), it is 0.89 trip per dwelling unit. This means that approximately two-thirds of all the plotted points are within 0.89 trip above or below the line, thus furnishing a measure of the reliability of estimates of trips from cars owned per dwelling. This also means that for the whole population it is probable that estimates of residents' trips per dwelling unit in each census tract in Washington, D. C., would be within ±0.89 trip of the true value in two-thirds of the cases when using average automobile ownership per dwelling unit per census tract as the predicting variable.

Relation of Trips to Population, Distance, and Income

The same analysis as illustrated in figures 2–4 was made for population density, distance from the central business district, and income per household. It is significant to note that trips decrease as population density increases. For comparison, the pertinent measures for each relation are presented in table 1. Notice that as the correlation coefficients decrease the errors of estimate increase. This is the usual case in regression analysis.

It would appear from the evidence thus far obtained that each of the four factors nfluence the number of trips made by residents in an urban area. However, this does not furnish any information about the interdependence of the variables. For example, how much does automobile ownership depend on income, population density, and distance from the city center? Further, does population density depend on income? Obviously, a high correlation would be obtained between population density and distance from the central business district. Intercorrelation among the variables used as predictors clouds the issue considerably. At this point in the investigation it is not known whether trips are a function of all of these variables working independently, or whether they are interacting together and in effect overlapping. Therefore, correlation coefficients were computed to give a perspective of their interaction. They are presented in table 2.

As was suspected, there is a high correlation between distance and population density and substantial correlations among all combinations of factors. To qualify technically as truly independent variables, all of the values in the table other than those shown in italics would have to be zero or so nearly so that the association could be ascribable to chance.

All of these variables were analyzed to produce one multiple regression equation[3] as follows:

$$X_1 = a + b_{12.345}X_2 + b_{13.245}X_3 + b_{14.235}X_4 + b_{15.234}X_5$$

Where:

X_1 = Residents' trips per dwelling unit.
X_2 = Automobile ownership per dwelling unit.
X_3 = Population density.
X_4 = Distance from the central business district.
X_5 = Income per household (index).
a = Intercept on the X_1 axis.
b = Multiple regression coefficient (similar to slope on a two-variable problem).

This technique requires the simultaneous solution of five equations to determine the five unknown quantities in the equation. It also makes possible the measurement of the relative importance of each variable when the effect of all the others acting simultaneously is taken into account. With the coefficients determined, the multiple regression equation is $X_1 = 4.33 + 3.89X_2 - 0.005X_3 - 0.128X_4 - 0.0120X_5$.

At first inspection it would appear that automobile ownership is the most important predictor of trips since it has the highest coefficient, and population density the least important, but comparison of the coefficients directly is not possible because of the different units of measurement of each of the independent variables. Beta coefficients that are

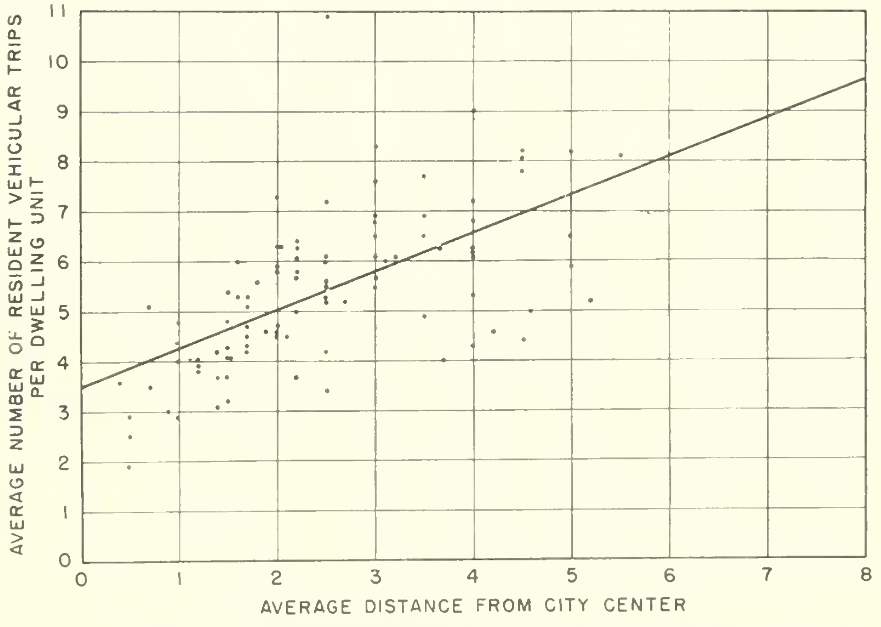

Figure 3.—Relation of resident vehicular trips to distance of residence from the central business district.

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[3] Example: $b_{12.345}$ should be read as "multiple regression coefficient of variables 1 and 2 with variables 3, 4, and 5 held constant."

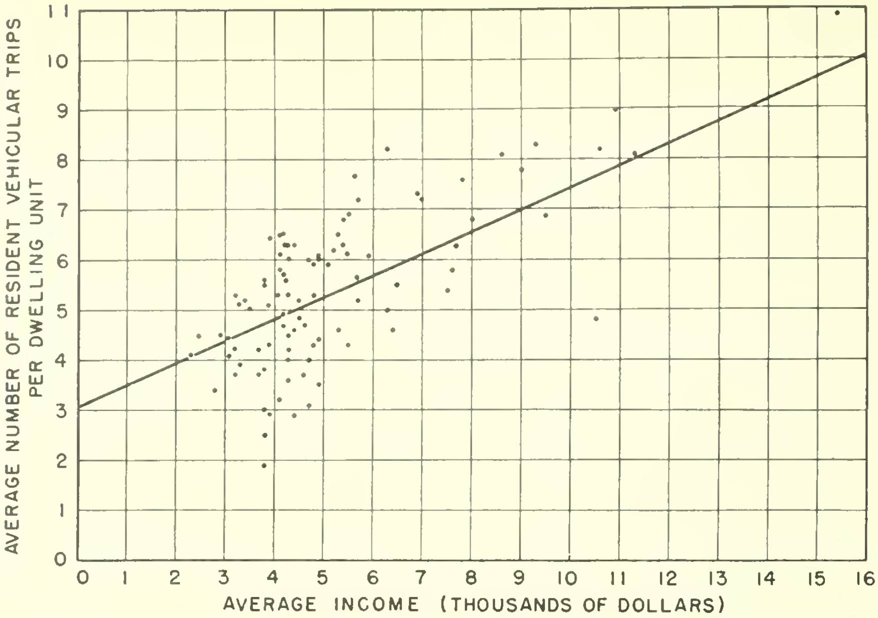

Figure 4.—Relation of resident vehicular trips to average income per household.

independent of units of measurement were computed to overcome this difficulty, thus permitting comparison in the same fashion as correlation coefficients.

Variable	*Beta coefficients*
Automobile ownership per dwelling unit	0. 700
Net population density	. 265
Distance from central business district	. 099
Income per household	. 018

Significance of Variables

The order of importance of the independent variables is now known, and the next step is to determine how much influence each has to the whole. The coefficient of multiple correlation of the four variables is 0.837 and the standard error of estimate is 0.87 trip. These are the indicators of the predictive reliability of all of the variables working together. Referring to table 1, it can be seen that automobile ownership alone had a correlation coefficient of +0.827 and a standard error of estimate of 0.89—almost as good as all variables taken together.

Another factor that casts doubt on the added value of population density, distance, and income as predictors of trips when used in conjunction with automobile ownership is that in the original two-variable analyses, all the regression coefficients (slopes) were positive with the exception of trips versus population density as would be expected; but in the multiple regression equation, all of the regression coefficients turned negative except automobile ownership. Such behavior is often the case when the net relation between two variables is very minor and the sample measure is easily tipped positively or negatively by chance variation.

A technique known as "analysis or partition of the variance" was employed to estimate the significance of the contribution of each of the independent variables. As stated before, the square of a correlation coefficient multiplied by 100 is the percentage of the variation in the dependent variable that can be explained by one or more independent variables. This concept is a valuable tool for further insight into the problem.

Since automobile ownership was the strongest independent variable, the variation attributable to it was taken out of the total variation in trips and tested against values in probability tables to determine whether it was significant or whether it could be due to chance.

If a coin were tossed a great many times, it would be expected that during very nearly one-half of the tosses the coin would fall heads and one-half tails. Actually, this means that the odds or probability on any given toss of the coin is one-half for a head and one-half for a tail. If two coins were tossed at a time, there would be four equally likely possibilities on any given toss. First, coin A may be heads and coin B tails; second, coin A may be tails and coin B heads; third, both coins may be heads; and fourth, both coins may be tails. Since there are four possible combinations in which the two coins may fall, the probability of both coins being heads is one-fourth, or the coin tosser may expect both to fall heads in approximately 25 percent of the tosses if chance were the only factor operating. The more times the coins were tossed, the greater the possibility of attaining the ideal. If both coins were actually heads in 50 percent of the tosses, the evidence would be very strong that chance was not the sole determinant and that some other influence was causing the difference in results.

The technique used in this investigation of factors affecting trips is merely an extension of the coin-tossing principle. It was decided to reject the possibility that a measure of variation differed from the value that would have been obtained had chance been the only factor operating if the odds were 99 to 1 against chance. In the area between 99 to 1 and 95 to 5, the results were classified as questionable or doubtful. If the odds are less than 95 to 5, the possibility of chance factors yielding such values for the measures of variation has been accepted and the variable has been rejected as not contributing anything significant to the problem.

In comparison to the coin-tossing experiment the standards selected are quite conservative. They are satisfactory for anyone willing to be wrong one time out of one hundred. These statements apply only to the data in hand. No inferences are made about the reliability of the sampling methods or the possible bias due to averaging the data on a census tract basis. This topic is discussed later in the article.

The increment of variation accounted for by the addition of population density, when the effect of automobile ownership is taken into account and held constant, was next de-

Table 1.—Correlation of residents' daily trips with selected variables, as measured by a study of 95 census tracts in Washington, D. C., on an average weekday in 1948

Independent variables	Trips per dwelling unit (dependent variable)		Predicting equation
	Correlation coefficient	Standard error of estimate	
Automobile ownership	+0. 827	0. 89	$X_1=2.88+4.60X_2$
Population density	−. 718	1. 10	$X_1=7.22-0.013X_3$
Income	+. 655	1. 20	$X_1=3.07+0.44X_5$
Distance from central business district	+. 575	1. 30	$X_1=3.55+0.74X_4$

Table 2.—Degree of correlation between the several variables, as measured by a study of 95 census tracts in Washington, D. C., on an average weekday in 1948

Variables	Correlation coefficients of each variable to all other variables				
	Trips per dwelling unit	Automobile ownership	Population density	Income per household	Distance from central business district
Trips per dwelling unit	----------	*+0. 827*	*−0. 718*	*+0. 655*	*+0. 575*
Automobile ownership	*+0. 827*	----------	−. 780	+. 786	+. 662
Population density	*−. 718*	−. 780	----------	−. 631	−. 825
Income per household	*+. 655*	+. 786	−. 631	----------	+. 448
Distance from central business district	*+. 575*	+. 662	−. 825	+. 448	----------

Table 3.—Source and amount of variation in trips per dwelling unit, as measured by a study of 95 census tracts in Washington, D. C., on an average weekday in 1948

Source of variation in trips per dwelling unit	Amount of variation in trips per dwelling unit	
	Automobile ownership data available	Automobile ownership data not available
	Percent	*Percent*
Gross amount explained by—		
Automobile ownership	68.34	----
Population density	----	51.55
Increment explained by—		
Income with population density held constant	----	14.00
Population density with automobile ownership held constant	4.29	----
Distance with automobile ownership and population density held constant	.94	----
Distance with population density and income held constant	----	.04
Income with automobile ownership, population density, and distance held constant	.04	----
Residual, unexplained	26.39	34.41
Total	100.00	100.00

termined and tested for significant difference from the value that would have been obtained if chance were the only factor operating. Then the increment due to distance from the central business district with the effect of both automobile ownership and population density held constant was tested for significance. This procedure was repeated for income with all other factors held constant. The analysis shows that automobile ownership alone is definitely significant. When the effect of automobile ownership is taken out, the increment added by population density is of doubtful significance. This means that the odds were about 95 to 5 that the additional variation accounted for could have been chance variation. The increments of distance and income did not prove to be significant, as indicated in the first column of table 3, and are probably due to chance variation only.

Since the increment of variation in trips explained by population density is of doubtful significance, but not weak enough to reject according to the standards set, a multiple regression equation including it with automobile ownership was developed to compare with the equation using ownership alone. This multiple regression equation, $X_1=3.80+3.79X_2-0.0033X_3$, is shown graphically in figure 5. The multiple correlation coefficient is 0.835 and the standard error of estimate is 0.87 trip. This may be compared with the results of using automobile ownership as the only predictor of trips ($X_1=2.88+4.60X_2$). In the latter instance, the correlation coefficient is +0.827 and the standard error of estimate is 0.89 trip. It can be seen from a comparison of the correlation coefficients and standard errors of estimate that population density adds very little to the predictive accuracy.

For cases where automobile ownership data are not available, the next best combination was determined. The order of importance of the independent variables, as reported in the second column of table 3, was found to be (1) population density, (2) income, and (3) distance from the central business district.

In this analysis, both population density and income were found to be significant and distance, not significant. The multiple regression equation for this combination, shown graphically in figure 6, is $X_1=5.49-0.0089X_3+0.227X_5$. The multiple correlation coefficient is 0.764 and the standard error of estimate is 1.02 trips. If population density alone is used as the predictor the regression equation is $X_1=7.22-0.013X_3$. The correlation coefficient is −0.718 and the standard error of estimate is 1.10 trips.

Each succeeding combination of independent variables used thus far has resulted in a decrease in the correlation coefficients and an increase in the standard errors. Table 4 shows all significant combinations in descending order of predictive accuracy.

As previously mentioned, data were available from the 1948 metropolitan area traffic survey on residents' trips and automobile ownership for the entire Washington, D. C., metropolitan area, but similar data on income and population density were unavailable for the suburban areas at the time of the study. The linear regression equation for trips and car ownership for the entire area in 1948 is $X_1=2.80+4.45X_2$, whereas for the District of Columbia, the equation is $X_1=2.88+4.60X_2$. The standard errors of estimate for the entire

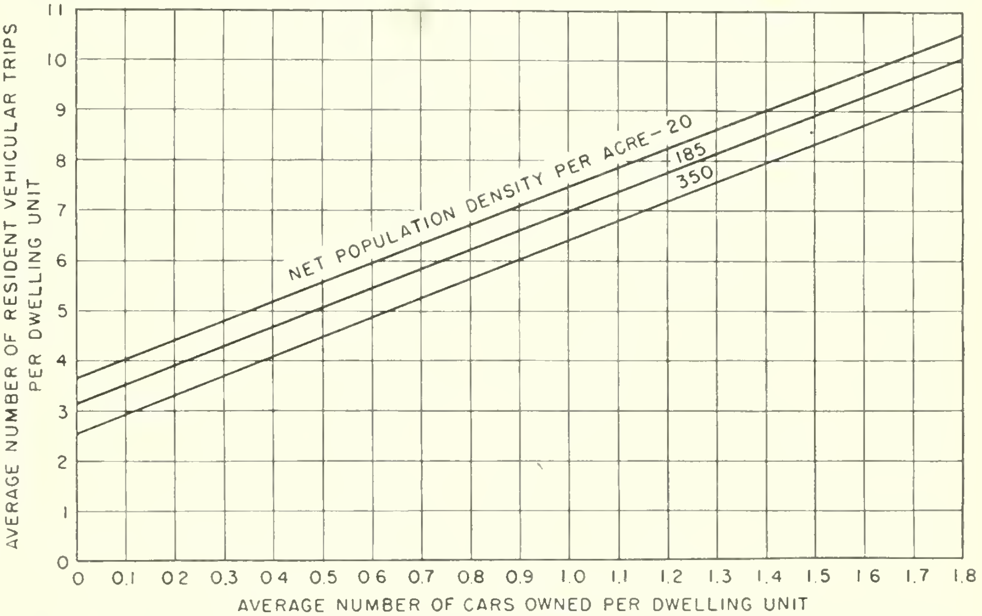

Figure 5.—Relation of resident vehicular trips to automobile ownership and population density.

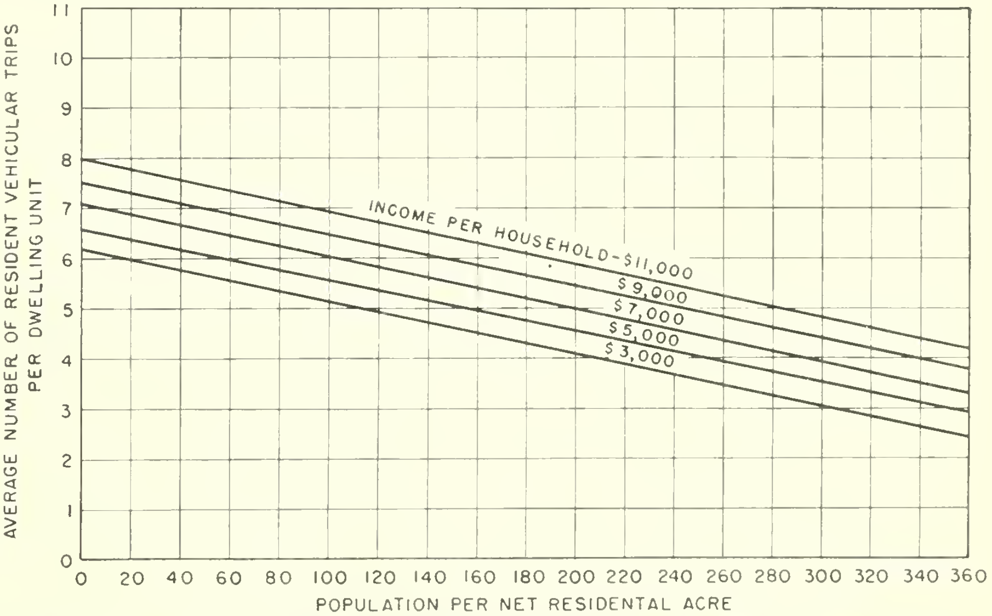

Figure 6.—Relation of resident vehicular trips to population density and average income per household.

Table 4.—Correlation of residents' daily trips per dwelling unit with various combinations of independent variables, as measured by a study of 95 census tracts in Washington, D. C., on an average weekday in 1948

Independent variables	Trips per dwelling unit (dependent variable)		Predicting equation
	Correlation coefficient	Standard error of estimate	
Automobile ownership and population density	0. 835	0. 87	$X_1=3.80+3.79X_2-0.0033X_3$
Automobile ownership	+. 827	. 89	$X_1=2.88+4.60X_2$
Population density and income	. 764	1. 02	$X_1=5.49-0.0089X_3+0.227X_5$
Population density	−. 718	1. 10	$X_1=7.22-0.013X_3$
Income	+. 655	1. 20	$X_1=3.07+0.44X_5$
Distance from central business district	+. 575	1. 30	$X_1=3.55+0.74X_4$

area and the District of Columbia are 0.91 and 0.89 trip per dwelling unit and the correlation coefficients are +0.815 and +0.827, respectively.

Factors Affecting Study

The methods presented here provide a means of evaluating very complex collections of data without personal bias. It should be pointed out, however, that the selection of the grouping units has a very definite effect on any statistical study. It is proper to form groupings whenever the individual units within a group are more nearly like each other with respect to the factors under consideration than they are like the units in any other group. Insofar as this is done effectively, just thus far is the analysis of the relation of variation between groups made more precise. In this study the interest is in the relation of number of trips to economic, geographic, and demographic factors. Census tracts establish groupings on the basis of these and other factors. Other groupings that may be more or less effective in attaining this purpose may lead to conclusions differing somewhat from those obtained in this study.

In all preceding analyses, emphasis was given to the relations of values of averages per dwelling unit per census tract. Had the interest been in studying the relation of values per dwelling unit, the measures of such a relation would have been smaller and the scatter greater than those obtained in this study, because the effect of many of the factors that were extraneous to this study could not have been eliminated partially or altogether by grouping. Likewise, it would have been even more difficult to study the relation of values for each dwelling unit rather than the relation of averages.

Comparison With More Recent Data

Preliminary results of an analysis of a 1955 followup traffic study strengthen the findings of the present study, as they apply to the relation between trips and automobile ownership per dwelling unit. The correlation coefficients and standard errors of estimate for 1948 were +0.827 and 0.89, respectively, as compared with +0.844 and 0.71 for 1955. The conclusion may be drawn that automobile ownership per dwelling unit is a good indicator for predicting the number of resident vehicular trips.

The application of the 1948 equation, $X_1=2.88+4.60X_2$ (X_1 represents trips per dwelling unit, and X_2, automobile ownership per dwelling), for estimating 1955 trips resulted in an across-the-board overestimate. This was primarily due to a change in procedure as to what constitutes a dwelling unit; the more recent study accounted for a greater number of dwelling units, such as roominghouses, military installations, and so forth, than did the earlier study.

Behavior of Riveted Connections in Truss-Type Members

**Digest of a report[1] by E. CHESSON, JR., Research Associate in
Civil Engineering, and W. H. MUNSE, Research Professor
of Civil Engineering, University of Illinois**

THIS article reports on a part of the cooperative research of the Bureau of Public Roads, Illinois Division of Highways, University of Illinois, and the Research Council on Riveted and Bolted Structural Joints.

Full-size tests under tensile loading were made on 16 specimens representing 8 variations of commonly used types of truss joints. There were two arrangements of laced-angle members and two of plate-and-angle members. Each was riveted to a pair of ½-inch gusset plates at each end which, in turn, were welded to large loading plates mounted in the testing machine. Rivet patterns represented common practice in joint design including an alternative pattern for one member. Three of the joint designs were tested with drilled holes and with punched holes.

By means of electric strain gages attached to the steel, the strains in different parts of the members were measured as the load was applied in increments up to failure. These tests, with mechanical dials and a whitewash coating to aid visual observation, revealed the distribution of load, points of weakness and initial failure, and the general nature of failure. The data were studied in relation to the tension, shear, bearing ratio, and theoretical joint efficiency.

Test loads ranged from 439,000 to 1,235,000 pounds depending upon the design of members. Tests of duplicate members agreed very well; in four cases the differences ranged from 1 to 2.4 percent, and in four cases from 6 to 8 percent. Several clear-cut indications are reported in the concluding remarks that follow.

Conclusions

1. Adherence to current design stresses does not necessarily insure a balanced design (i. e., a design in which, at ultimate, the member is likely to fail in either shear or tension); shear failures may be expected in long truss-type joints of "balanced design."

2. Large connections should be proportioned so that the distribution of rivets in a joint is similar to the distribution of areas connected by the rivets.

3. Members with drilled holes in the connections are more susceptible to shear failures than are similar punched specimens. In addition, the shear strength of the drilled member can be expected to be slightly smaller than that of the punched member.

4. Punched and drilled truss-type members of the same joint pattern and of ⅜- to ½-inch thick material may be expected to have approximately the same efficiency. This may be different for thicker materials or for loading conditions differing from these tests.

5. The use of lacing bars in tension members provides a secondary loading which may reduce the strength of the members. To reduce the likelihood of tensile failures at the lacing rivets, the edge distances at these rivets should be made as large as possible and the lacing bars as small as feasible.

6. Of the several design rules considered, the A. R. E. A. net-section rule appears to give the best agreement with the test efficiencies of these truss-type members.

7. In view of the lack of complete agreement between theoretical and test efficiencies and the unpredictable variations in the materials, it is doubted that complicated formulae for the design of tension members are justified. Because of the simplicity of application and our familiarity with the currently specified rule, it would seem desirable to retain the present net-section rule as a basis for design but to institute a suitable upper limit on efficiency or effective net section. Such a procedure would correct the most serious deficiency of the current specifications for tension members and would provide, for riveted connections, a predicted or theoretical efficiency which does not differ greatly from the test efficiency.

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[1] Proceedings of the American Society of Civil Engineers, Paper No. 1150, Journal of the Structural Division, Jan. 1957.

Annual Report of the Bureau of Public Roads, Fiscal Year 1956

The Annual Report of the Bureau of Public Roads for fiscal year 1956 is now available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 25 cents a copy.

The report covers a wide range of engineering, administrative, and research activities in the highway field. During the fiscal year, progress was notable because improvements of Federal-aid highways and of highways in general reached new high levels and also by reason of the action of the Federal Government in assuring completion of a 41,000-mile National System of Interstate and Defense Highways in the following 13 to 15 years.

The Federal-Aid Highway Act of 1956, approved June 29, 1956, authorized the greatest long-range roadbuilding program ever undertaken. It authorizes a total amount of $24,825 million for the Interstate System for the period beginning July 1, 1956, and ending June 30, 1969.

The $875 million provided by the Federal-Aid Highway Act of 1954 for fiscal year 1956, an increase of $300 million over the previous fiscal year, gave added momentum to the Federal-aid highway program. The apportionment of $875 million authorized for fiscal 1957 was made on August 9, 1955. This authorization and remaining balances of prior authorizations, together with State and local matching funds, financed the program carried forward during the year. Additional 1957 funds totaling $1,125 million, provided by the Federal-Aid Highway Act of 1956, were apportioned June 29, 1956.

During the fiscal year 1956, $687 million of Federal funds were used in the improvement of 23,828 miles of highways with a total cost of $1,306 million. Included were 6,673 miles of highways and 1,290 bridges on the Federal-aid primary highway system outside of cities, 957 miles of highways and 599 bridges on urban portions of the primary system, 15,289 miles of roads and 1,886 bridges on secondary roads, and 909 miles of highways in National parks, forests, parkways, and on flood-relief projects. Railway-highway grade crossings were eliminated at 212 locations, 27 inadequate structures were reconstructed, and 305 crossings were protected by installation of appropriate safety devices.

In addition to a discussion of new highway legislation, other subjects covered in this report which were not found in the previous edition include highway design standards, survey of right-of-way practices and procedures, roadside improvement, and organization and training.

The report notes that improvement of majo urban highways was of increasing concern t State highway departments. Programs wer approved during the year which included projects in urban areas for a total estimated cos of $555 million, the Federal contribution bein approximately $290 million. Work complete during the year cost $434 million for 957 mile of which the Federal contribution was $21 million. Planning and initiation of construction of expressways in urban areas was stimulated by recent emphasis given to the Interstate System.

In the field of research, the annual repor covers Bureau studies of highway finance highway transport, and the array of physica problems that are associated with highways.

Reviewing other significant developments the report points out that the year will b significant in highway history because it mark the end of discussion of what should be don to create an adequate system of main highway to serve the Nation and the adoption of a fir policy that will result in creation of such system in 13 to 15 years.

Lost Production in Highway Construction: A Motion Picture

The Bureau of Public Roads, U. S. Department of Commerce, recently announced the release of a new motion picture *Lost production in highway construction*. The film, based on extensive studies conducted by the Bureau, examines minor delays that affect production rates of key units of highway construction equipment, including power shovels, scrapers, hot-mix bituminous plants, and pavers.

As stressed in this motion picture, most minor delays in highway construction jobs are timed in seconds; but the seconds add rapidly into hours and lost dollars for the contractor, and higher roadbuilding costs to the public. Minor delays cannot be eliminated completely, but greater efficiency by the contractors will mean that roads will be put into service sooner and at lower cost. Contrasts in operation practices shown in the motion picture are very revealing.

The motion picture is a 16-mm. sound and color film with a running time of 30 minutes. Prints may be borrowed for showings by an responsible organization by request addresse to Visual Education, Bureau of Public Road Washington 25, D. C. There is no charg except for the express or postage fees. Requests for the film should be sent well i advance of the desired showing and alterna dates for showing should be given if possib Prompt return after each showing is necessar so that all requested bookings may be fulfille without delay.

Highway Statistics, 1955

The Bureau's HIGHWAY STATISTICS, 1955, the eleventh of the bulletin series presenting annual statistical and analytical tables of general interest on the subjects of motor fuel, motor vehicles, highway-user taxation, financing of highways, and highway mileage is now available.

The 174-page publication may be purchased from the Superintendent of Documents, Government Printing Office, Washington 25, D. C., at $1.00 a copy. The series of annual bulletins that are available from the Superintendent of Documents are indicated on the inside back cover of Public Roads.

U. S. GOVERNMENT PRINTING OFFICE: 1957

A list of the more important articles in PUBLIC ROADS may be obtained upon request addressed to Bureau of Public Roads Washington 25, D. C.

PUBLICATIONS
# of the Bureau of Public Roads

The following publications are sold by the Superintendent of Documents, Government Printing Office, Washington 25, D. C. Orders should be sent direct to the Superintendent of Documents. Prepayment is required.

ANNUAL REPORTS

Work of the Public Roads Administration:
  1941, 15 cents.
  1942, 10 cents.
  1948, 20 cents.
  1949, 25 cents.
Public Roads Administration Annual Reports:
  1943; 1944; 1945; 1946; 1947.
  (*Free from Bureau of Public Roads*)
Annual Reports of the Bureau of Public Roads:
  1950, 25 cents.
  1951, 35 cents.
  1952, 25 cents.
  1953 (out of print).
  1954 (out of print).
  1955, 25 cents.
  1956, 25 cents.

PUBLICATIONS

Bibliography of Highway Planning Reports (1950). 30 cents.
Braking Performance of Motor Vehicles (1954). 55 cents.
Construction of Private Driveways, No. 272MP (1937). 15 cents.
Criteria for Prestressed Concrete Bridges (1954). 15 cents.
Design Capacity Charts for Signalized Street and Highway Intersections (reprint from PUBLIC ROADS, Feb. 1951). 25 cents.
Electrical Equipment on Movable Bridges, No. 265T (1931). 40 cents.
Factual Discussion of Motortruck Operation, Regulation, and Taxation (1951). 30 cents.
Federal Legislation and Regulations Relating to Highway Construction (1948). Out of print.
Financing of Highways by Counties and Local Rural Governments: 1931–41, 45 cents; 1942–51, 75 cents.
General Location of the National System of Interstate Highways, Including All Additional Routes at Urban Areas Designated in September 1955. 55 cents.
Highway Bond Calculations (1936). 10 cents.
Highway Bridge Location, No. 1486D (1927). 15 cents.
Highway Capacity Manual (1950). $1.00.
Highway Needs of the National Defense, House Document No. 249 (1949). 50 cents.
Highway Practice in the United States of America (1949). 75 cents.
Highway Statistics (annual):
  1945 (out of print).
  1946, 50 cents.
  1947, 45 cents.
  1948, 65 cents.
  1949, 55 cents.
  1950 (out of print).
  1951, 60 cents.
  1952, 75 cents.
  1953, $1.00.
  1954, 75 cents.
  1955, $1.00.
Highway Statistics, Summary to 1945. 40 cents.
Highways in the United States, *nontechnical* (1954). 20 cents.
Highways of History (1939). 25 cents.
Identification of Rock Types (reprint from PUBLIC ROADS, June 1950). 15 cents.
Interregional Highways, House Document No. 379 (1944). 75 cents.
Legal Aspects of Controlling Highway Access (1945). 15 cents.
Local Rural Road Problem (1950). 20 cents.
Manual on Uniform Traffic Control Devices for Streets and Highways (1948) (*including 1954 revisions supplement*). $1.25.
  Revisions to the Manual on Uniform Traffic Control Devices for Streets and Highways (1954). *Separate*, 15 cents.

PUBLICATIONS (Continued)

Mathematical Theory of Vibration in Suspension Bridges (1950). $1.25.
Needs of the Highway Systems, 1955–84, House Document No. 120 (1955). 15 cents.
Opportunities in the Bureau of Public Roads for Young Engineers (1955). 25 cents.
Parking Guide for Cities (1956). 55 cents.
Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft (1943). $2.00.
Progress and Feasibility of Toll Roads and Their Relation to the Federal-Aid Program, House Document No. 139 (1955). 15 cents.
Public Control of Highway Access and Roadside Development (1947). 35 cents.
Public Land Acquisition for Highway Purposes (1943). 10 cents.
Public Utility Relocation Incident to Highway Improvement, House Document No. 127 (1955). 25 cents.
Results of Physical Tests of Road-Building Aggregate (1953). $1.00.
Roadside Improvement, No. 191MP (1934). 10 cents.
Selected Bibliography on Highway Finance (1951). 60 cents.
Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways, 1956: a reference guide outline. 55 cents.
Specifications for Construction of Roads and Bridges in National Forests and National Parks, FP-41 (1948). $1.50.
Standard Plans for Highway Bridge Superstructures (1956). $1.75.
Taxation of Motor Vehicles in 1932. 35 cents.
Tire Wear and Tire Failures on Various Road Surfaces (1943). 10 cents.
Transition Curves for Highways (1940). $1.75.

MAPS

State Transportation Map series (available for 39 States). Uniform sheets 26 by 36 inches, scale 1 inch equals 4 miles. Shows in colors Federal-aid and State highways with surface types, principal connecting roads, railroads, airports, waterways, National and State forests, parks, and other reservations. Prices and number of sheets for each State vary—see Superintendent of Documents price list 53.
United States System of Numbered Highways. 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

Bibliography on Automobile Parking in the United States (1946).
Bibliography on Highway Lighting (1937).
Bibliography on Highway Safety (1938).
Bibliography on Land Acquisition for Public Roads (1947).
Bibliography on Roadside Control (1949).
Express Highways in the United States: a Bibliography (1945).
Indexes to PUBLIC ROADS, volumes 17–19 and 23.
Title Sheets for PUBLIC ROADS, volumes 24–28.

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF FEBRUARY 28, 1957

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			CONTRACTS ADVERTISED CONSTRUCTION NOT STARTED			PROJECTS UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$29,258	$32,513	$25,863	264.4	$13,039	$8,418	120.1	$67,073	$39,874	832.0	$112,625	$74,155	1,216.5
Arizona	17,372	19,144	16,643	175.7	3,317	2,489	40.8	18,170	14,747	94.5	40,631	33,879	311.0
Arkansas	39,278	20,263	11,586	419.6	10,071	5,303	101.8	31,382	19,336	375.4	61,716	36,225	896.8
California	35,476	62,241	42,533	212.0	16,683	10,884	28.9	391,948	181,257	291.6	470,872	234,674	532.5
Colorado	32,733	19,397	13,342	149.4	5,542	3,363	48.6	30,929	21,594	170.3	55,868	38,299	368.3
Connecticut	39,159	13,198	8,169	7.6	9,677	8,123	10.3	9,622	4,843	16.1	32,497	21,135	34.0
Delaware	18,743	3,760	1,880	32.7	1,436	739	18.8	14,738	8,159	47.8	19,934	10,778	99.3
Florida	34,115	26,929	20,461	234.7	11,083	5,860	44.2	52,906	30,004	333.2	90,918	56,325	612.1
Georgia	65,305	33,818	18,444	415.7	8,389	4,078	53.4	84,409	48,705	788.2	126,616	71,227	1,257.3
Idaho	36,095	2,152	1,325	33.4	4,388	2,971	38.8	11,838	7,832	144.9	18,378	12,128	217.1
Illinois	36,361	89,656	66,031	627.3	44,277	30,744	107.4	145,285	101,933	583.7	279,218	198,708	1,318.4
Indiana	93,097	28,902	15,357	116.5	8,755	4,814	60.9	36,710	21,416	138.0	74,367	41,587	315.4
Iowa	9,246	65,454	47,908	1,109.1	26,591	17,975	280.1	32,928	21,896	676.2	124,973	87,779	2,065.4
Kansas	15,803	51,699	42,260	845.4	6,263	3,288	108.7	41,285	24,394	1,102.0	99,247	69,942	2,056.1
Kentucky	59,851	8,273	5,494	151.2	542	296	1.3	42,340	26,212	307.5	51,155	32,002	460.0
Louisiana	43,357	41,220	19,561	155.8	9,750	5,088	47.3	39,569	21,919	208.7	90,539	46,568	411.8
Maine	26,462	9,154	4,863	64.9	971	639	5.8	18,990	9,858	119.2	29,115	15,360	189.9
Maryland	7,863	27,669	16,667	69.8	16,312	12,608	20.5	50,342	31,867	146.2	94,323	61,142	236.5
Massachusetts	41,383	36,963	22,346	29.8	37,743	23,082	26.3	54,607	30,321	49.9	129,313	75,749	106.0
Michigan	54,031	74,143	55,001	453.0	37,298	22,331	70.3	66,033	40,003	279.5	177,474	117,335	802.8
Minnesota	36,254	27,897	20,686	640.6	9,841	6,142	113.0	58,964	39,295	731.7	96,702	66,123	1,485.3
Mississippi	12,284	51,946	38,183	788.3	14,331	11,190	128.0	31,378	17,999	574.5	97,655	67,372	1,490.8
Missouri	48,144	35,310	21,282	1,044.8	26,698	21,467	78.7	88,674	51,384	1,064.3	150,682	94,133	2,187.8
Montana	40,878	9,951	6,192	178.3	4,605	2,764	62.2	44,895	32,258	415.9	59,451	41,214	656.4
Nebraska	43,665	23,763	14,361	404.3	8,960	5,216	117.2	25,426	14,665	774.5	58,149	34,242	1,296.0
Nevada	35,656	4,550	3,798	119.6	1,941	1,673	6.0	11,404	9,621	134.7	17,895	15,092	260.3
New Hampshire	16,027	8,364	5,646	33.2	1,156	635	5.1	14,150	8,500	58.6	23,670	14,781	96.9
New Jersey	82,424	19,897	9,948	81.2	6,107	3,356	5.1	34,393	17,345	35.8	60,397	30,649	122.1
New Mexico	12,058	12,939	10,469	78.0	13,045	10,786	76.6	23,986	18,242	197.6	49,970	39,497	352.2
New York	132,775	51,553	32,099	129.9	78,390	50,846	61.6	308,147	172,168	353.2	438,090	255,113	544.7
North Carolina	66,488	35,887	23,143	440.9	13,788	7,169	117.5	54,397	27,495	657.8	104,072	57,807	1,216.2
North Dakota	18,269	25,481	19,111	1,282.7	8,957	6,628	153.9	17,420	9,624	818.4	51,858	35,363	2,255.0
Ohio	51,377	76,054	54,018	149.7	34,098	24,354	62.7	138,692	89,958	179.0	248,844	168,330	391.4
Oklahoma	32,072	46,256	31,264	469.8	15,922	9,330	121.7	47,048	26,875	419.0	109,226	67,469	1,010.5
Oregon	28,619	7,780	6,125	50.1	5,196	4,539	26.7	34,246	23,758	185.3	47,222	34,422	262.1
Pennsylvania	87,578	110,026	69,395	115.9	84,867	57,562	84.5	118,905	62,730	323.9	313,798	189,687	524.3
Rhode Island	5,010	10,625	8,198	14.9	2,773	2,496	.1	30,355	19,035	24.0	43,753	29,729	39.0
South Carolina	37,025	26,452	17,155	565.6	4,264	2,866	8.7	30,148	16,650	562.2	60,864	36,671	1,136.5
South Dakota	12,729	39,005	28,969	755.4	3,765	2,173	135.6	20,682	13,000	600.1	63,452	44,142	1,491.1
Tennessee	47,089	23,110	11,373	471.4	15,573	7,916	44.4	83,788	50,696	528.1	122,471	69,985	1,043.9
Texas	131,393	18,129	10,543	168.9	62,382	42,073	456.4	146,102	86,946	1,551.6	226,613	139,562	2,176.9
Utah	19,746	15,909	13,482	150.7	3,442	2,694	28.2	10,349	7,847	148.3	29,700	24,023	327.2
Vermont	12,848	7,753	5,388	37.9	1,276	718	4.0	13,773	8,926	59.3	22,802	15,032	101.2
Virginia	59,672	26,103	16,122	158.7	6,371	3,556	15.9	40,750	21,782	360.9	73,224	41,460	535.5
Washington	47,104	9,472	6,538	183.5	7,350	4,981	36.5	35,972	21,525	255.2	52,794	33,044	475.2
West Virginia	32,262	28,970	19,905	59.5	5,715	2,919	22.4	27,104	14,619	72.4	61,789	37,443	154.3
Wisconsin	63,521	15,935	9,109	214.0	10,736	5,546	61.3	56,978	34,360	314.9	83,649	49,015	590.2
Wyoming	3,377	27,245	22,444	238.7	5,882	4,416	59.6	24,115	17,924	270.3	57,242	44,784	568.6
Hawaii	6,599	2,082	1,041	8.8	3,324	1,659	5.1	4,741	2,247	8.3	10,147	4,947	22.2
District of Columbia	18,259	14,051	9,695	7.6	637	335	.5	9,850	7,430	.1	24,538	17,460	8.2
Puerto Rico	11,741	5,183	2,078	14.8	-	-	-	19,054	9,206	62.7	24,237	11,284	77.5
Alaska	15,074	-	-	-	-	-	-	-	-	-	-	-	-
TOTAL	2,003,005	1,484,226	1,003,494	14,625.7	733,519	481,098	3,333.5	2,846,990	1,660,280	18,447.5	5,064,735	3,144,872	36,406.7

VOL. 29, NO. 8

JUNE 1957

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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In this issue: A study of braking performance of passenger cars, trucks, and truck combinations.

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U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretary

BUREAU OF PUBLIC ROADS
BERTRAM D. TALLAMY, Administrator
CHARLES D. CURTISS, Commissioner

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Stopping Ability of Motor Vehicles Selected From the General Traffic

DIVISION OF HIGHWAY TRANSPORT RESEARCH

BUREAU OF PUBLIC ROADS

Reported [1] by F. WILLIAM PETRING, Highway Transport Research Engineer

This article presents the results of tests conducted during 1955 on more than 1,200 vehicles selected at random from the general traffic. It compares past and present levels of brake performance, and shows that improvements in the brake performance of most vehicle types since 1949 have been small. Current performance levels are reported according to vehicle type, gross weight, vehicle capacity, and axle load.

The improvement in general levels of brake performance was smaller between 1949 and 1955 than it was in the earlier period, 1942–49. Only the 3-axle truck-tractors with 2-axle semitrailers and the truck-tractor-semitrailer and full-trailer combinations showed substantially better braking in both periods. The small amount of improvement made by most vehicle types since 1949 indicates that the wide range in stopping abilities which exists among the various types will not be appreciably reduced in the near future. Consequently, this range must be taken into account in highway design, vehicle regulation, and driver training.

The observed levels of performance in stops from 20 miles per hour for commercial vehicles operating in the general traffic stream with normal loads averaged about 25 feet for 2-axle trucks with a manufacturer's gross vehicle weight rating of 10,000 pounds or less. The average levels generally ranged from 35 to 45 feet for other 2-axle trucks, from 40 to 50 feet for the 3-axle trucks, from 45 to 55 feet for truck-tractor-semitrailer combinations, and from 55 to 65 feet for trucks with full trailers and truck-tractor-semitrailer and full-trailer combinations.

The tests also revealed that many commercial vehicles, except for the very light 2-axle trucks, were inadequately braked in proportion to the loads carried on individual axles. However, individual tests proved that vehicles with axle loads as high as 22,000 pounds can be adequately braked.

The percentages of vehicles tested which met the Uniform Vehicle Code specifications for brake-system application and braking distance present a little brighter picture. Some 92 percent of the passenger cars and 84 percent of each of two classifications of 2-axle trucks were able to meet their respective distance specifications of 25, 30, and 40 feet, respectively. About 80 percent of the 3- and 4-axle truck-tractor-semitrailer combinations and 3-axle trucks, and 38 to 64 percent of the largest vehicle combinations complied with their code specification of 50 feet.

Also of interest is the degree of compliance with the deceleration requirement of the Uniform Vehicle Code. Well over 99 percent of the passenger cars met the deceleration specification of 17 feet per second per second. At the same time, all very light 2-axle trucks and 94 percent of other 2-axle trucks met the 14 feet per second per second requirement. So did 76 to 85 percent of the truck-tractor-semitrailer combinations and 3-axle single-unit trucks, 51 percent of the trucks with full trailers, and 69 percent of the truck-tractors with semitrailers and full trailers.

AT THE request of representatives of both Government and industry, the Bureau of Public Roads in 1941 initiated a rather broad program of research on motor-vehicle brakes. Although the program was halted by World War II, one important phase of the work—tests of motor vehicles selected from the everyday traffic—was completed in 1944. In 1947, the program was reinstituted and field work was performed from November 1948 to September 1951. A comprehensive report [2] on the findings of the postwar research was published in 1954.

As a result of the information published by the Bureau of Public Roads, the National Committee on Uniform Traffic Laws and Ordinances revised its Uniform Vehicle Code [3] to provide a more realistic brake performance requirement. Since that time seven States have revised their brake performance requirements, patterning them after the Uniform Vehicle Code.

This article, which covers tests conducted during 1955, provides current information on the braking ability of various types of motor vehicles in the general traffic. It brings up to date similar information presented in earlier reports and compares present levels of brake performance with those determined by earlier tests.

As in previous brake studies, the Bureau of Public Roads was responsible for collection and analysis of the data, but each State in which tests were conducted provided assistance with the field work. Personnel for weighing the vehicles and recording data were furnished by the Maryland State Roads Commission, the California Division of Highways, and the Michigan State Highway Department. Uniformed personnel, who stopped the vehicles to be tested and kept traffic moving smoothly at the test sites, were provided by Maryland State Roads Commission Truck Patrol, California Highway Patrol, and Michigan State Police. Test apparatus used in the field work was furnished by General Motors Proving Ground, McCormick Engineering Co., and the Institute of Transportation and Traffic Engineering of the University of California.

Purposes of Study

The primary purposes of the study were as follows: (1) To show the levels of brake performance that prevail for the various types of vehicles currently found on the highways; (2) to bring up to date, information used as a basis for formulating brake performance regulations for such vehicles; (3) to provide current motor-vehicle brake performance data which may be used in establishing highway design standards; (4) to show the trend in levels of brake performance of the various types of vehicles using the highways; and (5) to focus attention on the existing brake performance situation, particularly with respect to the effect of load and vehicle type.

---

[1] This article was presented at the National Transportation Meeting of the Society of Automotive Engineers, New York City, October 10, 1956.

[2] *Braking performance of motor vehicles*, by Carl C. Saal and F. William Petring. Published by the Bureau of Public Roads, 1954.

[3] *Uniform vehicle code*, revised 1954. National Committee on Uniform Traffic Laws and Ordinances, Washington, D. C.

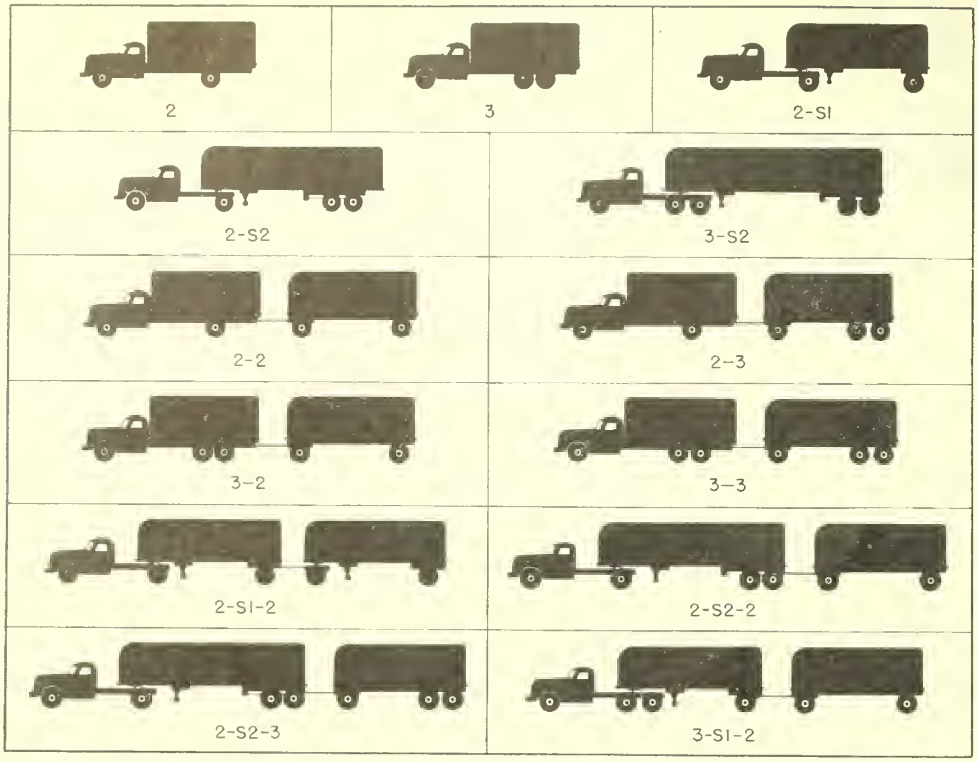

Figure 1.—Commercial vehicle types as designated by code based on axle arrangement.

Terminology

Certain terms used in this article must be thoroughly understood in order to have a clear conception of the results. Definitions of these terms follow:

Brake-system application time or distance.—The time elapsed or distance traveled between the instant or point at which the driver starts to move the braking controls and the instant or point of first retardation by the brakes.

Braking time or distance.—The time elapsed or distance traveled between the instant or point of first retardation by the brakes and the instant or point at which the vehicle comes to rest.

Brake-system application and braking time or distance.—The time elapsed or distance traveled between the instant or point at which the driver starts to move the braking controls and the instant or point at which the vehicle comes to rest. (This definition is identical to the one for "vehicle stopping time or distance" which appears both in the Bureau of Public Roads 1954 report, Braking Performance of Motor Vehicles, and in the 1956 Society of Automotive Engineers Handbook.)

Table 1.—Number of vehicles tested by State, type, and axle arrangement in 1955 and in 1949

Type of vehicle [1]	Number of vehicles tested							
	1955				1949			
	Maryland	Michigan	California	Total	Maryland	Michigan	California	Total
Passenger cars	104	101	105	310	99	112	100	311
Single-unit trucks:								
2-axle	134	138	128	400	132	148	87	367
3-axle	20	28	25	73	19	4	23	46
Subtotal	154	166	153	473	151	152	110	413
Truck-tractors with semitrailers:								
2-S1	69	39	21	129	132	89	25	246
2-S2	93	43	17	153	17	44	37	98
3-S2	2	29	35	66	------	2	55	57
Subtotal	164	111	73	348	149	135	117	401
Trucks with trailers:								
2-2	------	9	7	16	------	6	3	9
2-3	------	------	1	1	------	------	2	2
3-2	------	13	33	46	------	4	12	16
3-3	------	------	7	7	------	------	27	27
Subtotal	------	22	48	70	------	10	44	54
Truck-tractors with semitrailers and trailers:								
2-S1-2	------	9	35	44	------	18	32	50
2-S2-2	------	5	2	7	------	6	3	9
2-S2-3	------	2	------	2	------	1	------	1
3-S1-2	------	------	1	1	------	------	------	------
Subtotal	------	16	38	54	------	25	35	60
Other vehicle combinations:								
Passenger-car driveaway-towaway	------	3	------	3	------	------	------	------
Housetrailer factory tow-away	------	5	------	5	------	------	------	------
Subtotal	------	8	------	8	------	------	------	------
Grand total	422	424	417	1,263	399	434	406	1,239

[1] For an explanation of the code used, see p. 179.

Brake force buildup time or distance.—The time elapsed or distance traveled between the instant or point of first retardation by the brakes and the instant or point maximum braking force is attained.

Pedal reserve.—The distance, in inches, between the floorboard or mat and the back of the pedal at the completion of a stop.

Swerving.—An uncontrollable lateral movement of the vehicle when the brakes are applied, which is involuntary on the part of the driver.

Deceleration.—The rate of reduction of the vehicle speed in feet per second per second.

Maximum deceleration.—The greatest deceleration measured during the stop regardless of the length of time it is sustained.

Manufacturer's gross vehicle weight rating.—The weight, in pounds, of the truck chassis with lubricants, water, and full tank or tanks of fuel, plus the weights of cab or driver's compartment, body, special chassis and body equipment, and payload as authorized by the chassis manufacturer.

Vehicle capacity.—In the case of single-unit trucks, vehicle capacity is the maximum gross vehicle weight rating; for combination units, it is the gross combination weight recommended by the vehicle chassis manufacturer for a given truck-tractor or truck to be used in combination with semitrailers or trailers.

Brake system types

Hydraulic.—Brake shoes are actuated by hydraulic brake cylinders operated with hydraulic line pressure developed by a pedal-operated hydraulic brake master cylinder.

Mechanical.—Brake shoes are actuated by a cam or wedge operated by a cable or rod linked to the brake pedal.

Vacuum-booster hydraulic.—Brake shoes are actuated by a hydraulic brake wheel cylinder operated with hydraulic line pressure develope by a vacuum-powered master cylinder or a vacuum hydraulic power unit.

Air-booster hydraulic.—Brake shoes ar actuated by a hydraulic brake wheel cylinde operated with hydraulic line pressure develope by an air-powered master cylinder or an ai hydraulic power unit.

Vacuum-mechanical.—Brake shoes are actuated by a cam or wedge operated by a vacuum brake chamber through a mechanica linkage.

Air-mechanical.—Brake shoes are actuate by a cam or wedge operated by an air brak chamber through a mechanical linkage.

Scope of Study

Tests to determine the brake-system application and braking distances of a representative number of vehicles were conducted o passenger cars and commercial vehicles selected at random from the general traffi The vehicles were stopped on the highway b a uniformed policeman, weighed and de scribed, and then subjected to emergency stops from a speed of 20 miles per hour. The test was not compulsory, and each driver clearly understood that punitive action would not be taken. The tests were conducted during 1955 on heavily traveled highways in Maryland and Michigan at the same sites used in a similar study in 1949, and in California at a site in the same general area as that used in 1949.

Table 1 shows the number of vehicles, by type and axle arrangement, tested in each State in 1949 and 1955. A similar series of tests, conducted during 1941 and 1942, included 907 passenger cars, 1,403 single-unit trucks, 1,597 truck-tractors with semitrailers, 216 trucks with full trailers, and 36 truck-tractors with semitrailers and full trailers. This early study was conducted at 11 locations in 10 States of wide geographical distribution. In order to maintain a continuing record of the levels of brake performance of the various types of vehicles in use on the highways, it is planned to conduct similar tests about every 5 years at locations as near as practicable to the three used in the 1949 and 1955 tests.

Conclusions

The following conclusions are based on the type and number of vehicles tested, and on the test procedures employed:

1. Except for two vehicle types, the improvement in the general levels of brake performance of motor vehicles selected from the general traffic was considerably smaller between 1949 and 1955 than it was between 1942 and 1949. The 3-axle truck-tractor with 2-axle semitrailer combinations and the truck-tractor-semitrailer and full-trailer combinations each showed a perceptible improvement of about the same magnitude in both periods. In contrast, for all other vehicle types, the improvement was appreciable in the earlier period while it was negligible in the later period.

2. In view of the small magnitude of improvement in brake-system application and braking distance made by most vehicle types since 1949, it appears that the wide range in stopping ability of the various vehicle types will not be significantly reduced in the near future. It follows that this wide range must be recognized in highway design, vehicle regulation, and driver training activities.

3. From 38 to 64 percent of the largest combination vehicle types and about 80 percent of the 3- and 4-axle truck-tractor-semitrailer combinations and 3-axle trucks were capable of complying with the Uniform Vehicle Code specification of a 50-foot stop from 20 miles per hour; 84 percent of each of the two classifications of 2-axle trucks and 92 percent of the passenger cars were able to comply with their respective distance specifications of 40, 30, and 25 feet.

4. Almost all passenger cars and 2-axle trucks, 76 to 85 percent of the 3-axle trucks and truck-tractor-semitrailer combinations, 69 percent of the truck-tractors with semitrailers and full trailers, and 51 percent of the trucks with full trailers were capable of meeting their respective Uniform Vehicle Code specifications for deceleration when tested with a pendulum-type decelerometer.

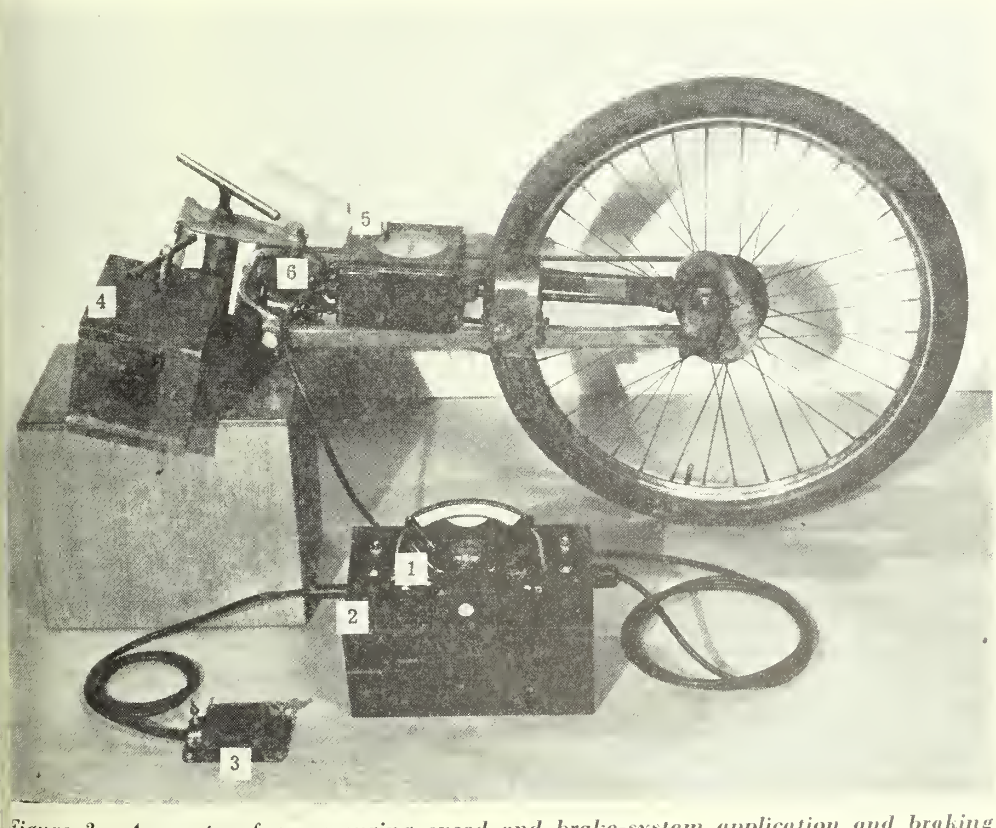

Figure 2.—Apparatus for measuring speed and brake-system application and braking distance: speedometer (1), control box (2), pedal switch (3), running-board clamp (4), odometer (5), and generator (6).

5. The observed average levels of brake performance in stops from 20 miles per hour for commercial vehicles, operating in the general traffic with normal loads, were about 25 feet for 2-axle trucks with a manufacturer's gross vehicle weight rating of 10,000 pounds or less, and ranged generally from 35 to 45 feet for other 2-axle trucks, from 40 to 50 feet for 3-axle trucks, from 45 to 55 feet for 3-, 4-, and 5-axle truck-tractor-semitrailer combinations, and from 55 to 65 feet for trucks with full trailers and truck-tractor-semitrailer and full-trailer combinations.

6. Many of the commercial vehicles, except very light 2-axle trucks, were found to be inadequately braked in proportion to the loads carried on individual axles. This condition may result from inadequate maintenance, axle loads heavier than those for which the brake components were designed, and/or poor selection of brake components. However, individual tests proved that vehicles with axle loads as high as 22,000 pounds can be adequately braked.

7. A substantial percentage of passenger cars and single-unit trucks selected from the general traffic showed a tendency to swerve—a condition that may be very dangerous in making emergency stops from normal speeds.

Vehicle Type Code

For convenience in reference, commercial vehicle types were assigned a code. Each digit represents the number of axles of a vehicle or of one unit of a vehicle combination. A combination symbol consisting of 2 or 3 parts separated by hyphens indicates a vehicle combination. The first digit of a combination symbol represents the power unit. An "S" in the second part of a combination symbol indicates a semitrailer, and the power unit, of course, is a truck-tractor. A digit appearing without an "S" in either the second or third position in a combination symbol represents a full trailer.

2 =2-axle single-unit truck
3 =3-axle single-unit truck
2-S1 =2-axle truck-tractor with 1-axle semitrailer
2-S2 =2-axle truck-tractor with 2-axle semitrailer
3-S2 =3-axle truck-tractor with 2-axle semitrailer
2-2 =2-axle truck with 2-axle trailer
2-3 =2-axle truck with 3-axle trailer
3-2 =3-axle truck with 2-axle trailer
3-3 =3-axle truck with 3-axle trailer
2-S1-2=2-axle truck-tractor with 1-axle semitrailer and 2-axle trailer
2-S2-2=2-axle truck-tractor with 2-axle semitrailer and 2-axle trailer
2-S2-3=2-axle truck-tractor with 2-axle semitrailer and 3-axle trailer
3-S1-2=3-axle truck-tractor with 1-axle semitrailer and 2-axle trailer

Illustrations of the various types of commercial vehicles tested and their respective code designations appear in figure 1.

The Test Sites

All of the tests were conducted on dry, smooth pavement, and each test section was approximately level and located on a tangent. The pavements at the Michigan and California test sites were portland cement concrete, and that at the Maryland site was a bituminous mat over portland cement concrete. In order to determine whether results obtained on the bituminous surface would be comparable with those obtained on portland cement concrete

Table 2.—Classification of vehicles tested by vehicle type, capacity, and brake type

Vehicle type and capacity	Brake type [1]	Number of vehicles tested in—			
		Maryland	Michigan	California	Total
Passenger cars	H	92	82	98	272
	VBH	11	19	6	36
	M	1	--------	1	2
2-axle trucks:					
Very light	H	40	23	40	103
	VBH	--------	3	--------	3
	M	--------	--------	1	1
Light	H	6	13	5	24
	VBH	50	68	59	177
	ABH	--------	--------	1	1
	M	--------	--------	1	1
Medium	H	--------	1	--------	1
	VBH	21	25	15	61
	ABH	3	--------	--------	3
	AM	9	1	2	12
	M	--------	1	--------	1
Heavy	VBH	1	1	1	3
	ABH	1	--------	--------	1
	AM	4	1	3	8
3-axle trucks:					
Light	VBH	4	--------	1	5
	ABH	--------	5	--------	5
	Other	2	1	1	4
Medium	VBH	2	7	9	18
	ABH	1	2	--------	3
	AM	3	3	1	7
	Other	1	--------	--------	1
Heavy	VBH	--------	--------	3	3
	AM	7	10	10	27
2-axle truck-tractors with 1-axle semitrailers:					
Light	VBH-VM	5	13	2	20
	ABH-VM	4	--------	1	5
	AM-AM	--------	--------	1	1
Medium	VBH-VM	2	13	7	22
	ABH-AM	4	1	--------	5
	AM-AM	20	5	5	30
	Other	6	1	1	8
Heavy	AM-AM	25	5	4	34
	AM-VM	3	1	--------	4
2-axle truck-tractors with 2-axle semitrailers:					
Light	VBH-VM	--------	--------	1	1
	Other	1	--------	1	2
Medium	VBH-VM	--------	3	1	4
	ABH-AM	--------	3	--------	3
	AM-AM	21	13	1	35
	Other	--------	5	--------	5
Heavy	AM-AM	71	18	12	101
	AM-VM	--------	1	1	2
3-axle truck-tractors with 2-axle semitrailers:					
Medium	AM-AM	2	9	2	13
	ABH-AM	--------	1	--------	1
Heavy	AM-AM	--------	19	32	51
	VBH-VM	--------	--------	1	1
3-axle trucks with 2-axle trailers:					
Medium	AM-AM	--------	2	--------	2
Heavy	AM-AM	--------	11	33	44
Other trucks with trailers:					
Light	VBH-VM	--------	--------	1	1
	Other	--------	--------	2	2
Medium	VBH-VM	--------	--------	1	1
	AM-AM	--------	--------	1	1
Heavy	AM-AM	--------	9	10	19
2-axle truck-tractors with 1-axle semitrailers and 2-axle trailers (heavy)	AM-AM-AM	--------	9	34	43
	Other	--------	--------	1	1
Other truck-tractors with semitrailers and trailers:					
Medium	VBH-VM-VM	--------	--------	1	1
Heavy	AM-AM-AM	--------	7	2	9
Other vehicle combinations:					
Passenger-car driveaway-towaway	H	--------	2	--------	2
	VBH	--------	1	--------	1
Housetrailer factory towaway	H-E	--------	5	--------	5

[1] Brake types: *H*=hydraulic, *M*=mechanical, *E*=electric, *VM*=vacuum-mechanical, *AM*=air-mechanical, *VBH* vacuum-booster hydraulic, and *ABH*=air-booster hydraulic.

surfaces, a few preliminary tests were run before final selection of the Maryland test site. Locked-wheel stops were made with a passenger car on the bituminous-covered section of pavement, later selected for the brake tests, and on a section of the same highway having a good portland cement concrete surface. Results of these tests showed that the sliding coefficient of friction for stops made from 20 miles per hour was almost identical at the two sites and within the range of values normal for good portland cement concrete pavement surfaces. The site with the bituminous surfacing was selected for this study because it was the one used for the 1949 study, and also because of certain safety features. In the earlier study, the concrete pavement had not been resurfaced.

Each of the three test sites was located on a section of 4-lane divided highway. The lane in which tests were conducted was closed off with rubber traffic cones placed at approximately 75-foot intervals for a distance of about one-half mile. All through-traffic was diverted to the other lane. Advance signs warned drivers to slow down, indicated that brake tests were in progress, and directed through-traffic into the open lane. A patrolman stationed at the approach to the test lane directed vehicles to be tested into the weighing area and kept other traffic moving smoothly through the open lane.

The Instrumentation

The key measurement in the study was brake-system application and braking distance.[4] A test wheel, shown in figure 2, was used to measure this distance in feet and to indicate vehicle speed in miles per hour. The apparatus was so arranged that the instant the driver put his foot on a switch, which was securely fastened to the face of the brake pedal on the test vehicle, an electrical circuit was completed which started the distance-measuring device. The circuit was maintained by a holding relay until released by the observer after the vehicle had come to rest. Thus, the distance was measured from the point at which the driver started to move the braking control to the point at which the vehicle came to rest. Whenever a hand control valve was to be used during the brake test, a small fingerlike switch was attached to the valve control handle and included in the circuit in conjunction with the pedal switch. Distance was thereby measured from the point of first contact with a braking control.

Since brake-system application and braking distance varies with speed at a significant rate, an accurate determination of speed also was essential. The vehicle speed was indicated directly by a voltmeter calibrated in miles per hour. The voltmeter, which was held in the lap of the observer, was wired to a belt-driven generator mounted on the frame of the test wheel.

In order to insure accuracy of the data, the test wheel was calibrated at frequent intervals. To calibrate the speedometer, the time required to travel a measured mile at a constant speed of 20 miles per hour, as indicated by the test wheel, was measured with a stopwatch and compared with the computed time required to travel a mile at that speed. In every instance the actual time required to travel the mile varied from the computed time by less than 1 percent. The distance indicated by the test wheel was also checked on the measured mile and found to vary from the true distance by less than 0.2 percent.

The accuracy of the test wheel in measuring brake-system application and braking distance was also verified by means of an electric detonator mounted on the bumper of the vehicle. The detonator, when fired, ejected a chalk capsule which marked the pavement directly beneath it. The firing mechanism was wired to the same pedal switch used to actuate the distance-measuring device on the test wheel. The instant the driver starte to move the braking control the gun fire causing the chalk capsule to mark the pavement, and simultaneously the test whe started to measure the distance traveled.

After the vehicle came to rest, the distanc was measured from a point on the pavemen directly below the detonator to the poin where the gun was fired, which was clearl indicated by the mark on the pavemen This distance was compared with the distanc indicated by the test wheel. Periodic test made in this manner showed a variation o less than 2 percent between the two metho of measurement.

In addition to the brake-system applicatio and braking distance, measurements wer made of the maximum deceleration as ind cated by a portable decelerometer. The d celerometer used was a popular make of the pendulum type. A moving scale on the instrument is graduated to read percent braking efficiency, which is actually the percentage of one-G deceleration. For example, a reading of 80 percent on the instrument would be a deceleration of 0.80×32.2 feet per second per second or 25.8 feet per second per second. Movement of the scale is actuated by and is proportional to movement of a damped pendulum. When the vehicle is moving at a uniform speed, the pendulum will assume a vertical position. However, if the vehicle speed is reduced, as by the application of brakes, the pendulum tending to move on at the same speed will swing forward. The tangent of the angle through which the pendulum moves forward away from the vertical is directly proportional to the deceleration. The instrument holds the moving scale at the maximum value attained until released by the observer.

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[4] See definition, p. 178

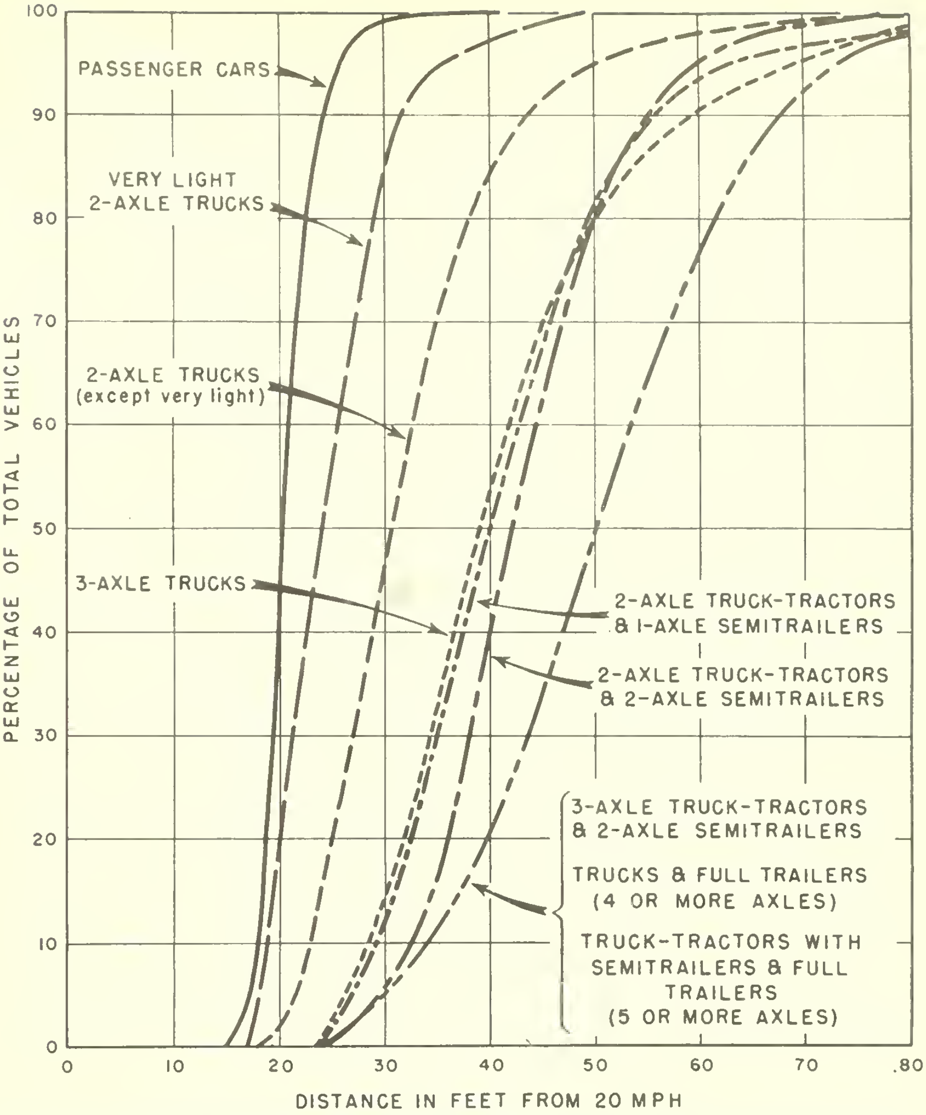

Figure 3.—Cumulative frequency distribution of minimum brake-system application and braking distances of motor vehicles selected from the general traffic.

Vehicle Sample

At the test location in each of the three States, approximately 300 commercial vehicles and 100 passenger cars were selected from traffic and tested. The commercial vehicles were selected to produce a sample about equally distributed between single-unit trucks and combination vehicles. Selectivity was exercised also to provide a wide range of vehicle makes, gross weights, capacities, and brake types. The passenger cars were chosen to represent a wide range of makes and year models. Vehicles loaded in an unsafe manner, and those whose drivers objected to participating in the test, were not included. However, only a very small percentage of the drivers asked to be excused.

Test Operations

Each vehicle selected for test was stopped by a uniformed officer as it approached the test section. The driver was directed to a spot where a crew was stationed to weigh the vehicle, record pertinent data, and install the apparatus for measuring brake performance. Each axle of the vehicle to be tested, including passenger cars, was weighed. In Maryland the vehicles were weighed on loadometers set in pits in a parking area adjacent to the highway; in California and Michigan, they were weighed on State-owned platform scales located adjacent to the test section.

Each vehicle tested was assigned a test number for use in coordinating the field data. Four separate forms were used to tabulate the following information: (1) axle weights, (2) vehicle characteristics, (3) skid data, and (4) brake performance and incidental data observed during the road tests.

Vehicle characteristics were recorded during the weighing operation. This description included the vehicle make, model, year model, type, manufacturer's gross weight rating, brake type, number of axles without brakes, total number of axles, and whether the vehicle was loaded or empty. When the weighing of the vehicle and the recording of vehicle characteristics had been completed, the apparatus for measuring brake performance was installed and the road tests were conducted. The regular driver of the vehicle, under the direction of an observer who rode beside him, made the emergency stops.

The road tests for each vehicle consisted of at least three emergency stops from a speed of 20 miles per hour. The observer noted, in the case of a commercial vehicle, whether there was a hand control valve and if it was used in making the stops. He also noted whether there was a valve for limiting the pressure applied to the front wheel brakes and, if so, the valve setting used in making the stops. After each stop the observer recorded the brake-system application and braking distance as indicated by the test wheel, the decelerometer reading, the brake pedal reserve, whether the vehicle swerved, and whether the clutch was engaged or disengaged during the stop.

As previously mentioned, all traffic was excluded from the lane in which the tests were being conducted. However, as an added precaution in case some vehicle should enter the test lane, each vehicle being tested was followed by an observer in another vehicle. The space between the two vehicles was held to the distance necessary to prevent other traffic from entering. A large sign reading DANGER—BRAKE TESTS—SUDDEN STOPS was attached to the rear of the observer's vehicle, as shown on the cover page, and a flashing red warning light was mounted on top. After each stop the second observer measured and recorded the lengths of any skid marks left by the test vehicle.

Classification of Vehicles

One of the first steps in the analysis of the test results was the classification of data: first by vehicle type, second by vehicle capacity groups for each vehicle type, and finally by brake types for each vehicle capacity group. Table 1 shows the number of vehicles tested in each State, both in 1949 and 1955, by vehicle type and axle arrangement. Table 2 shows by capacity groups and by brake types, the number of vehicles of each type tested in 1955.

All commercial vehicles were segregated into vehicle capacity groups on the basis of the chassis manufacturer's gross vehicle weight or gross combination weight rating.

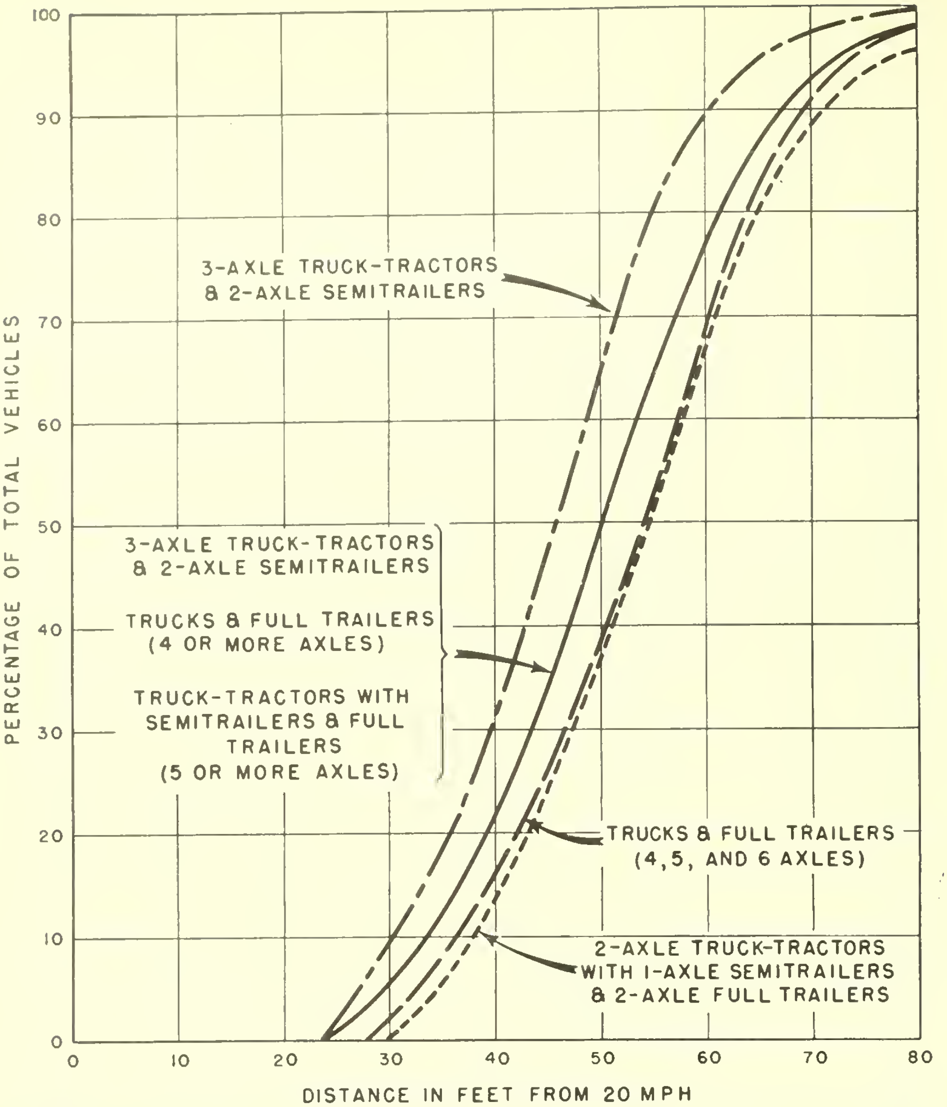

Figure 4.—Cumulative frequency distribution of minimum brake-system application and braking distances of the larger combination vehicles.

Single-unit trucks were classified into four groups (very light, light, medium, and heavy); and the combination vehicles, three groups (light, medium, and heavy). The range of gross weight ratings for single-unit and combination vehicles is shown in table 3.

In many instances the vehicle chassis manufacturer's recommended maximum weight for a truck or truck-tractor, used in combination with trailers, was not available. When this occurred the combination was classified as light, medium, or heavy on the basis of the power unit when used as a single-unit truck.

The most common brake types for the single-unit trucks were hydraulic and vacuum-booster hydraulic. The truck-tractor-semitrailer combinations were most commonly equipped with vacuum-booster hydraulic brakes on the tractor and vacuum-mechanical on the semitrailer or air-mechanical brakes on both units of the combination. The truck and full-trailer combinations and truck-tractor-semitrailer and full-trailer combinations were generally equipped with air-mechanical brakes on all units. As in the the 1949 study, air-mechanical brakes predominated on the heavy capacity commercial vehicles.

Table 3.—Classification of vehicles by capacity groups

Vehicle capacity group	Manufacturer's gross weight rating for—	
	Single-unit vehicles	Combination vehicles
	Pounds	*Pounds*
Very light	10,000 and under	----------------
Light	10,001–16,000	27,000 and under
Medium	16,001–24,000	27,001–44,000.
Heavy	Over 24,000	Over 44,000.

The greatest percentage of the passenger cars were equipped with hydraulic brakes However, the increasing popularity of power brakes was evident in that approximately 12 percent of the passenger cars tested were equipped with vacuum-booster hydraulic brakes. The percentage of passenger cars observed in the study equipped with mechanical brakes indicates that very few cars so equipped remain on the highways. Out of the total sample of 310 passenger cars, only 2 were equipped with mechanical brakes. One of the two was a postwar vehicle of foreign manufacture and the other a prewar American make.

It is apparent from table 2 that the limited size of the sample and the nature of its distribution by vehicle type, capacity group, and brake type precludes comprehensive comparisons of brake performance by brake types However, the sample was adequate to compare the performance of the two predominating brake types observed on passenger cars, light single-unit trucks, and medium 2-axle truck-tractors with 1-axle semitrailers.

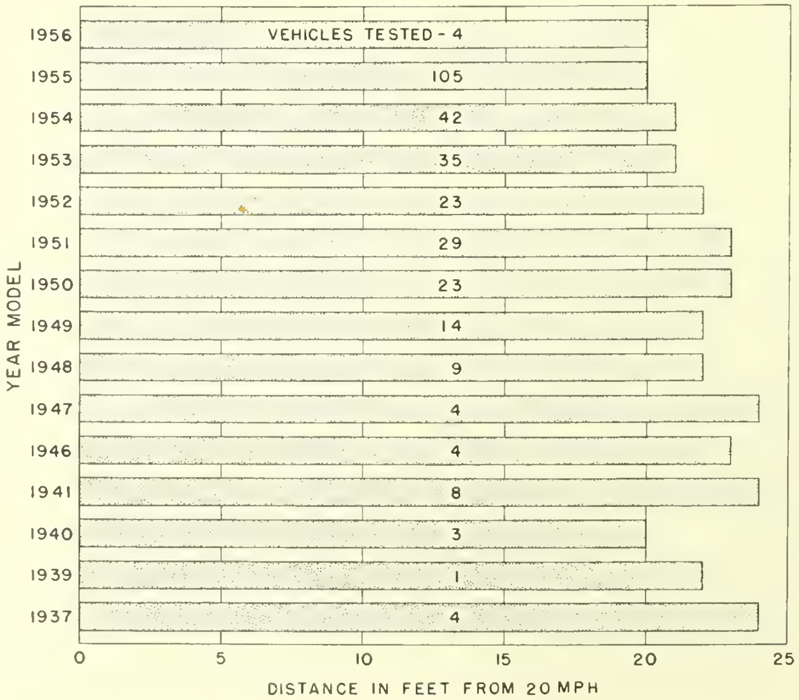

Figure 5 (Right).—*Variation of average brake-system application and braking distances with year model for passenger cars equipped with hydraulic brakes and vacuum booster hydraulic brakes.*

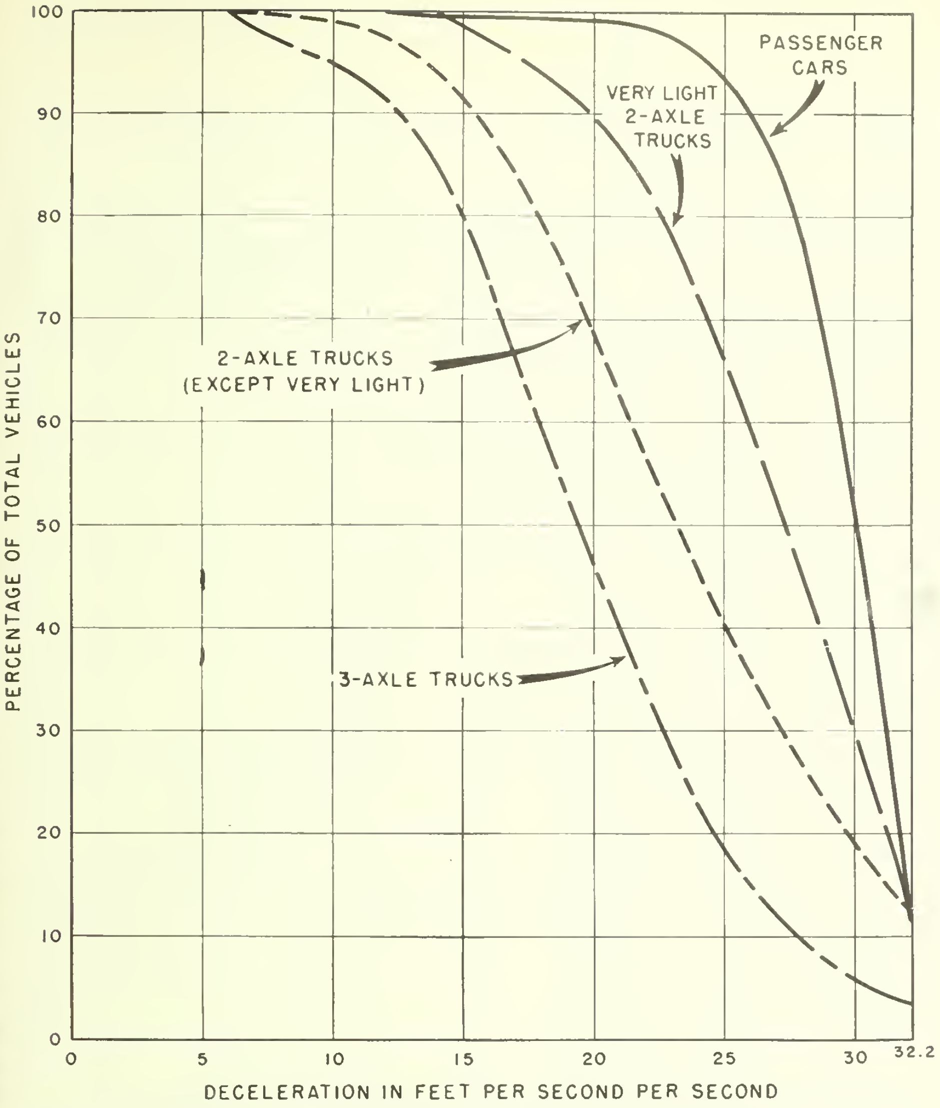

Figure 6.—Percentage of single-unit vehicles capable of a given or greater deceleration.

Consistency of Distance Data

As previously mentioned, the road tests of each vehicle consisted of three emergency stops from 20 miles per hour. In many instances, the first stop and sometimes the second did not represent the best possible braking performance of the vehicle, as frequently the driver was at first hesitant in making a true emergency stop, or did not understand that the shortest possible stop was desired. Since the length of the level test section was usually too short to make additional runs and it was not practical to turn vehicles around on the highway, the tests were not repeated.

In view of the fact that each of the three runs did not always represent the best brake performance of the vehicle being tested, an average of the three runs would not correctly reflect the true level of performance. Therefore, the minimum brake-system application and braking distance recorded for each vehicle was used in the analysis of the test results. This procedure also was followed in analyzing the results of the two previous studies. On the average there was a variation of slightly over 4 percent between the minimum or the maximum and the average brake-system application and braking distance observed for the three stops.

General Levels of Performance

The levels of brake performance of the various types of vehicles selected from the general traffic in 1955 are indicated in figures 3–8. In figures 3–5, where brake-system application and braking distance is the criterion of performance, the best performance is indicated by the shortest distance. But in figures 6–8, where deceleration is the criterion, the best performance is indicated by the highest deceleration. In studying the general results presented in the illustrations, it must be kept in mind that the sample of vehicles of each type is made up of various gross weights, capacities, brake types, and conditions of maintenance.

Brake-system application and braking distance

Figures 3 and 4 reveal quite clearly the differences in stopping ability of the various types of vehicles. The curves indicate the percentage of vehicles of each type that can stop in a given distance or less from a speed of 20 miles per hour. For example, figure 3 shows that the 50-percentile values for the various types of vehicles were as follows: Passenger cars, 20 feet; very light 2-axle trucks, 24 feet; other 2-axle trucks, 31 feet; 3-axle trucks, 39 feet; 2-axle truck-tractors with 1-axle semitrailers, 40 feet; 2-axle truck-tractors with 2-axle semitrailers, 42 feet; and all other combination vehicles, 50 feet. It is of particular interest to note that the curves shown in figure 3 converge at the lower percentile values. Thus, improvement in the levels of braking performance of all vehicle types will bring the performance levels of the individual types closer together. It is obvious that the greatest possibilities for improvement exist with the larger commercial vehicles.

In figure 4 the performance of the group of larger combination vehicles, which had the poorest stopping ability of any of the groups represented in figure 3, is shown for the three major types of combinations. For purposes of comparison, a composite curve (solid line) is included for the entire group. It is readily apparent that the 3-axle truck-tractors with 2-axle semitrailers, as a group, performed better than either the trucks with full trailers or the 2-axle truck-tractors with 1-axle semitrailers and 2-axle full trailers. Figure 4 also shows that there was little difference in the braking performance of the latter two groups of combination vehicles.

The average performance of passenger cars equipped with hydraulic and vacuum-booster hydraulic brakes is shown by year model in figure 5. It is evident that age had little influence on the stopping ability of this group of vehicles. On the average the 1955 and 1956 model cars were the best performers, requiring 20 feet to stop from 20 miles per hour. Also, the average stopping ability of the three 1940 model passenger cars tested equalled that of the 1955 and 1956 cars. Although the 1947, 1941, and 1937 model passenger cars were the poorest performers, they required, on the average, only 24 feet to stop. Since the sample was quite small for passenger cars of 1948 vintage and older, the results may not precisely reflect the stopping ability of each individual year model of the vehicles in this group. However, the total sample of these older vehicles was large enough to produce reliable results when considered as a group. The average brake-system application and braking distance for the 1948 model and older passenger cars was 23 feet from 20 miles per hour.

Deceleration

Although measurement of brake-system application and braking distance from a known speed provides the most reliable indication of braking ability, it is also significant to consider maximum deceleration, since brake performance enforcement in many jurisdictions is based entirely on measurement of deceleration. The curves shown in figures 6–8 indicate the decelerating abilities of the various types of vehicles tested. It should be kept in mind that the decelerations measured were not sustained throughout the stops, but were the maximum decelerations indicated during the stops.

Each curve shown in the graphs indicates the percentage of vehicles of a particular type which reached a maximum deceleration of a given or greater value. For example, figure 6 shows that 50 percent of each of the vehicle types represented by the four curves were capable of the following or greater decelerations, expressed in feet per second per second: passenger cars, 30.1; very light 2-axle trucks, 27.4; other 2 axle trucks, 23.2; and 3-axle trucks, 19.4. Similarly, figure 7 shows that 50 percent of each of three types of truck-tractor-semitrailer combinations were capable of the following or greater decelerations, expressed in feet per second per second: 2-axle truck-tractors with 1-axle semitrailers, 18.5; 2-axle truck-tractors with 2-axle semitrailers, 17.2; and 3-axle truck-tractors with 2-axle semitrailers, 16.8. Figure 8 shows that 50 percent of the trucks with full trailers and truck-tractors with semitrailers and full trailers were capable of decelerations of at least 14.2 and 17.1 feet per second per second, respectively. It is evident from figures 6–8 that there is a wide range in the decelerating ability of the various types of vehicles, just as it is evident from figures 3–4 that there is a wide range in their brake-system application and braking distances. Again, it is apparent that the smaller vehicles are capable of better performance.

Changes in Performance Levels Since 1942 and 1949

Between 1942 and 1949 there was considerable improvement in the brake performance of all types of motor vehicles selected from the general traffic. However, between 1949 and 1955 the improvement in brake performance of the various types of vehicles, with two exceptions, was quite small. Table 4 summarizes the 15-, 50-, and 85-percentile values of brake-system application and braking distance for stops made from 20 miles per hour by the various vehicle types tested in 1942, 1949, and 1955. This general summary again includes vehicles of various gross weights, capacities, brake types, and conditions of maintenance. It will be noted from the table that there was appreciable improvement between 1942 and 1949 in the braking performance at all three levels for each of the vehicle types tested. The greatest improvement, as would be expected, was at the 85-percentile level. Although the improvement at the 50- and 15-percentile levels was progressively smaller, it was still significant for most vehicle types.

Between 1949 and 1955 there was some improvement at the 85-percentile level in the brake performance of each vehicle type even though it was very small for some types. However, at the 50- and 15-percentile levels the performance of some vehicle types in 1955 was slightly below that of 1949. Only two vehicle types showed continuous improvement at all three performance levels. The truck-tractor-semitrailer with full-trailer combinations showed moderate improvement, and the 3-axle truck-tractors with 2-axle semitrailers showed substantial improvement at all three performance levels in both 1949 and 1955. Of course, it is evident that there was more room for improvement in these vehicles than there was for some of the other types.

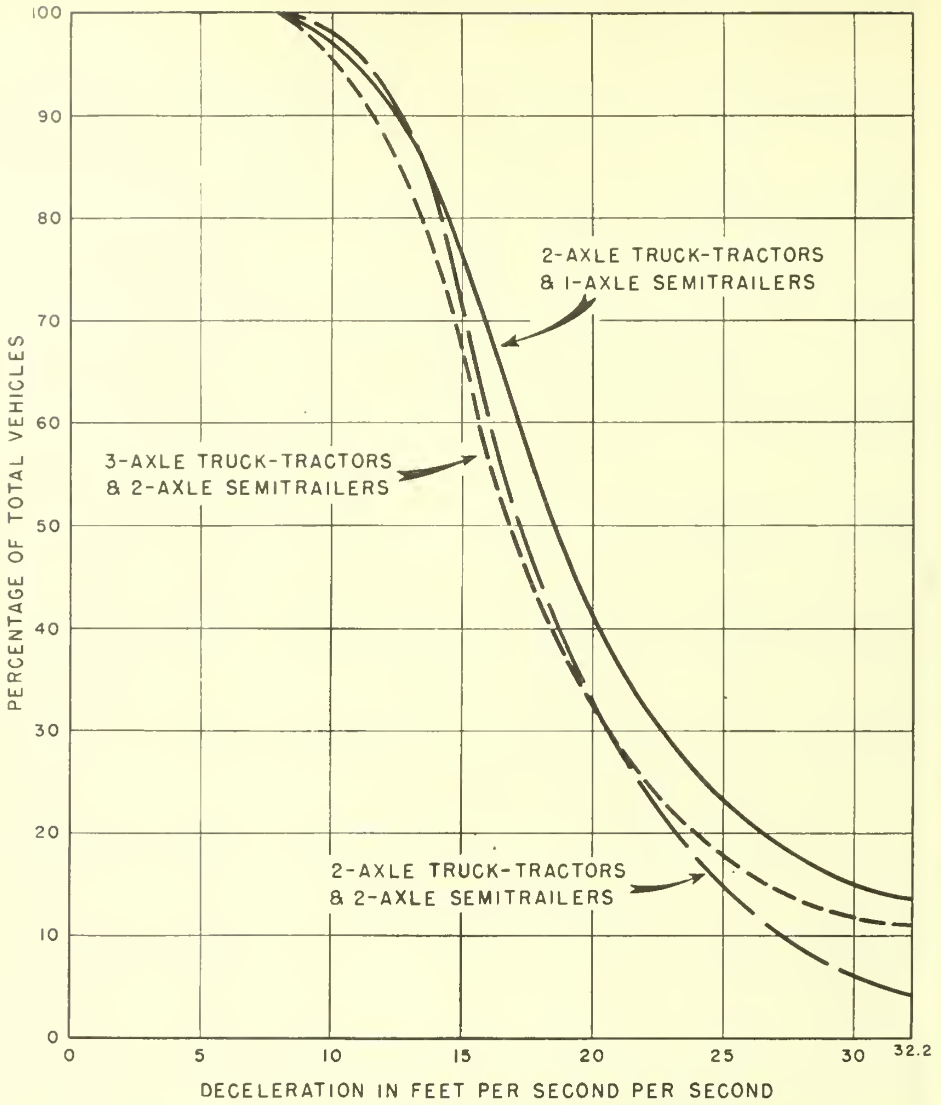

Figure 7.—Percentage of truck-tractor-semitrailer combinations capable of a given or greater deceleration.

The improvement in the 3-axle truck-tractors with 2-axle semitrailers is illustrated throughout the range of performance levels by comparing the cumulative frequency distribution curves, shown in figure 9, for the 1955 tests with similar curves plotted from data obtained from the tests conducted in 1949 and 1942. One of the most revealing observations to be made from these curves concerns the percentages of vehicles tested which could stop within 50 feet—the present standard of the Uniform Vehicle Code for combinations of commercial vehicles. In 1942, only 17 percent of these vehicles could stop within 50 feet; but in 1949, 36 percent could stop in 50 feet, and in 1955, 64 percent could stop within 50 feet or less. Between 1949 and 1955, 3-axle truck-tractors with 2-axle semitrailers showed a greater increase than any other type in the percentage of vehicles capable of complying

Table 4.—Comparison of brake-system application and braking distances from 20 miles per hour for various vehicle types tested in 1942, 1949, and 1955

Vehicle type	Brake-system application and braking distance in feet								
	15-percentile level			50-percentile level			85-percentile level		
	1942	1949	1955	1942	1949	1955	1942	1949	1955
Passenger cars	21	17	18	25	21	20	42	26	23
2-axle trucks	28	23	22	40	28	29	64	41	39
3-axle trucks	40	26	30	54	37	39	81	56	54
2-axle truck-tractors with 1-axle semitrailers	38	30	31	52	39	40	75	53	52
2-axle truck-tractors with 2-axle semitrailers	43	34	35	61	43	42	83	60	52
3-axle truck-tractors with 2-axle semitrailers	49	42	33	61	53	46	82	69	57
Trucks with full trailers (4, 5, and 6 axles)	48	39	39	65	52	54	87	70	66
Truck-tractors with semitrailers and full trailers (5, 6, and 7 axles)	47	45	41	62	59	53	82	73	67

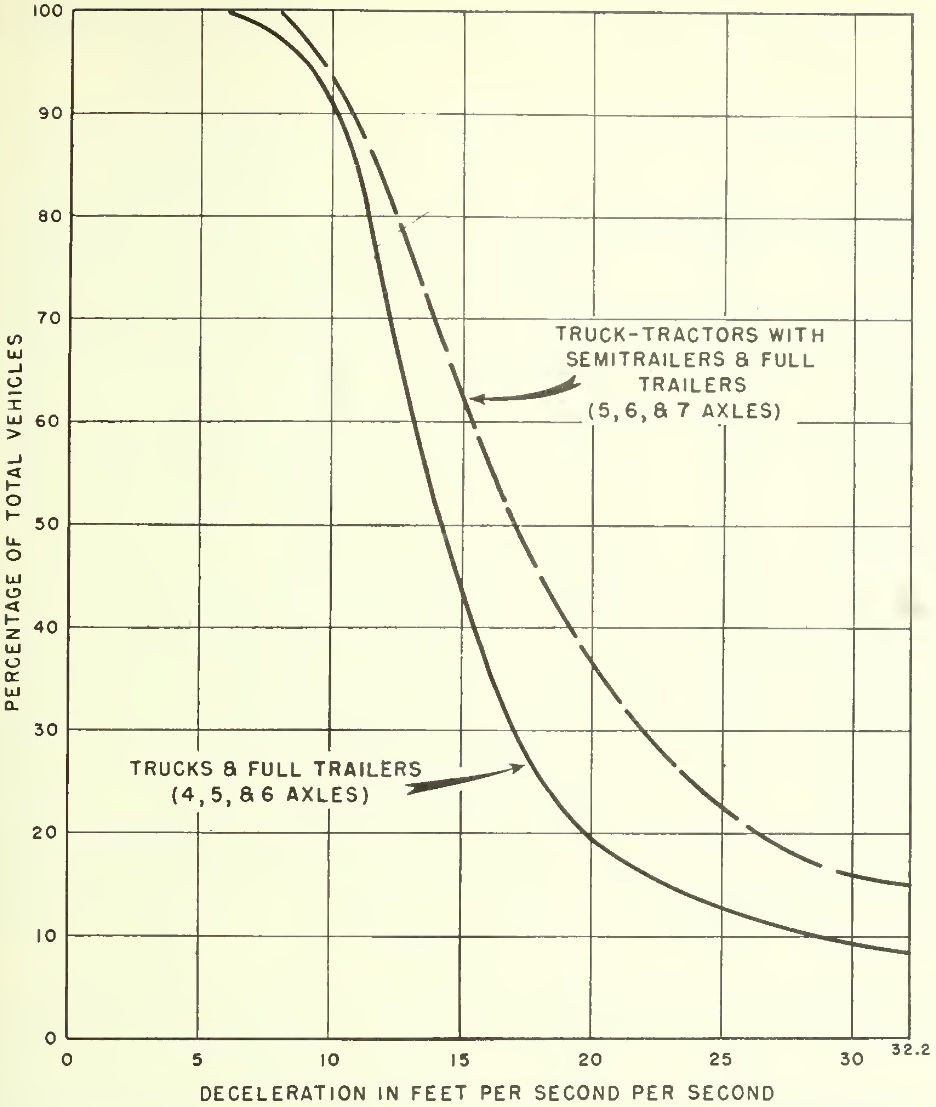

Figure 8.—Percentage of trucks with full trailers and truck-tractor-semitrailer and full-trailer combinations capable of a given or greater deceleration.

with the present requirement of the Uniform Vehicle Code. Such gains lead to the belief that there are excellent opportunities for considerable improvement in the braking performance of other commercial vehicle types.

If one considers the ability of the various vehicle types to comply with brake performance regulations, the improvements between 1949 and 1955 appear a little more encouraging than indicated by table 4. Table 5 shows by vehicle type the percentage of vehicles tested, both in 1949 and 1955, which could meet the present Uniform Vehicle Code requirement for brake-system application and braking distance for stops made from 20 miles per hour. In studying the results presented in table 5, it should be kept in mind that the requirement varies for the different vehicle types. For example, the requirement for very light 2-axle trucks (those having a manufacturer's gross vehicle weight rating of 10,000 pounds or less) is 30 feet; for 2-axle trucks other than very light, 40 feet; and for all other types of commercial vehicles, 50 feet.

Table 5.—Vehicles tested in 1949 and 1955 capable of meeting the Uniform Vehicle Code requirement, as revised in 1954, for brake-system application and braking distance from 20 miles per hour

Vehicle type	Requirement for brake-system application and braking distance	Vehicles capable of meeting requirement	
		1949	1955
	Feet	*Percent*	*Percent*
Passenger cars	25	83	92
Very light 2-axle trucks	30	76	84
Other 2-axle trucks	40	84	84
3-axle trucks	50	78	80
2-axle truck-tractors with 1-axle semitrailers	50	81	81
2-axle truck-tractors with 2-axle semitrailers	50	73	80
3-axle truck-tractors with 2-axle semitrailers	50	36	64
Trucks with full trailers (4, 5, and 6 axles)	50	44	38
Truck-tractors with semitrailers and full trailers (5, 6, and 7 axles)	50	22	41

As previously mentioned, the greatest percentage increase of vehicles capable of meeting the requirement between 1949 and 1955 was made by the 3-axle truck-tractors with 2-axle semitrailers; next in order were truck-tractors with semitrailers and full trailers. The latter type improved from 22 to 41 percent able to meet the requirement, an increase of 19 percent. Moderate increases in the percentage of vehicles able to comply with their respective requirements were made by passenger cars, very light 2-axle trucks, and the 2-axle truck-tractors with 2-axle semitrailers, which gained 9, 8, and 7 percent, respectively.

The 3-axle trucks showed a small improvement of 2 percent, whereas 2-axle trucks having a manufacturer's gross vehicle weight rating of over 10,000 pounds and the 2-axle truck-tractors with 1-axle semitrailers showed no change. The only vehicle type which lost ground with respect to its ability to meet the present Uniform Vehicle Code requirement for brake-system application and braking distance was the truck and full-trailer combination. In 1949, 44 percent of the sample of this vehicle type could comply, but in 1955, only 38 percent could meet the requirement.

It was not possible to compare the decelerating abilities of vehicles tested in 1949 and 1955, because deceleration was not measured in the 1949 studies of vehicles selected from the general traffic. However, it is of particular interest to view the results of deceleration studies made in 1955 with respect to Uniform Vehicle Code specifications. Table 6 shows the percentages of vehicles tested in 1955 which could meet their respective requirements for deceleration. The deceleration specified for passenger cars is 17 feet per second per second, whereas for all commercial vehicle types it is 14 feet per second per second.

Well over 99 percent of the passenger cars, all of the very light 2-axle trucks, and almost 94 percent of the other 2-axle trucks were able to meet the requirements for deceleration when measured with a commercially available portable decelerometer. From 76 to 85 percent of the truck-tractor-semitrailer combinations and 3-axle single-unit trucks, and 51 to 69 percent of the trucks with full trailers and truck-tractors with semitrailers and full trailers could comply with the 14 feet per second per second specification for deceleration.

A comparison of the results given in tables 5 and 6 shows that a greater percentage of vehicles of each type could comply with the Uniform Vehicle Code specification for deceleration than could comply with the corresponding specification for brake-system application and braking distance. However, in examining the two tables it should be kept in mind that the specifications for deceleration and for brake-system application and braking distance correspond to each other mathematically only for passenger cars and very light 2-axle trucks.

In making the foregoing comparisons of the performance of commercial vehicles tested in 1942, 1949, and 1955, the amount of improvement should be related to the weight of the vehicles in each sample of data. In other words, if the weights of the 3-axle truck-tractors with 2-axle semitrailers were considerably less in 1955 than in 1949, the improvement might be attributed to weight rather than to the braking system. From the tabulation of average gross weights given in table 7 for various types of vehicles, it is seen that average gross weights were generally higher in 1949 than in 1942, and similarly weights recorded in 1955 generally exceeded those in 1949. However, the reverse is true of the 2-axle trucks. The average weights of the 2-axle trucks tested were progressively lighter in 1949 and 1955.

The small decrease in average gross weights of the 3-axle truck-tractors with 2-axle semitrailers in 1955 may possibly account for a part of their improvement in stopping ability. However, since the weight decrease was relatively small, only 3,200 pounds and less than 6 percent, the greater part of the improvement may be attributed to more efficient braking systems.

The increases in the average gross weights of truck and full-trailer combinations, particularly those having 5 or 6 axles, can at least partly explain the poorer showing of these vehicles with respect to their ability to meet the present standard of the Uniform Vehicle Code. However, results of the controlled brake tests of commercial vehicles, which were reported in 1954,[5] indicate that considerable improvement can be made in the stopping ability of these vehicle combinations. The average-weight increases for the four types of truck and full-trailer combinations were as follows: 2-axle truck with 2-axle trailer, 2,800 pounds; 2-axle truck with 3-axle trailer, 11,400 pounds; 3-axle truck with 2-axle trailer, 17,000 pounds; and 3-axle truck with 3-axle trailer, 14,000 pounds. These increases amount to 6½, 28, 36, and 28 percent, respectively. The 3-axle trucks with 2-axle full trailers showed the greatest increase in average gross weights of any of the vehicle types tested and were by far the predominating type of truck and full-trailer combination encountered in the 1955 tests.

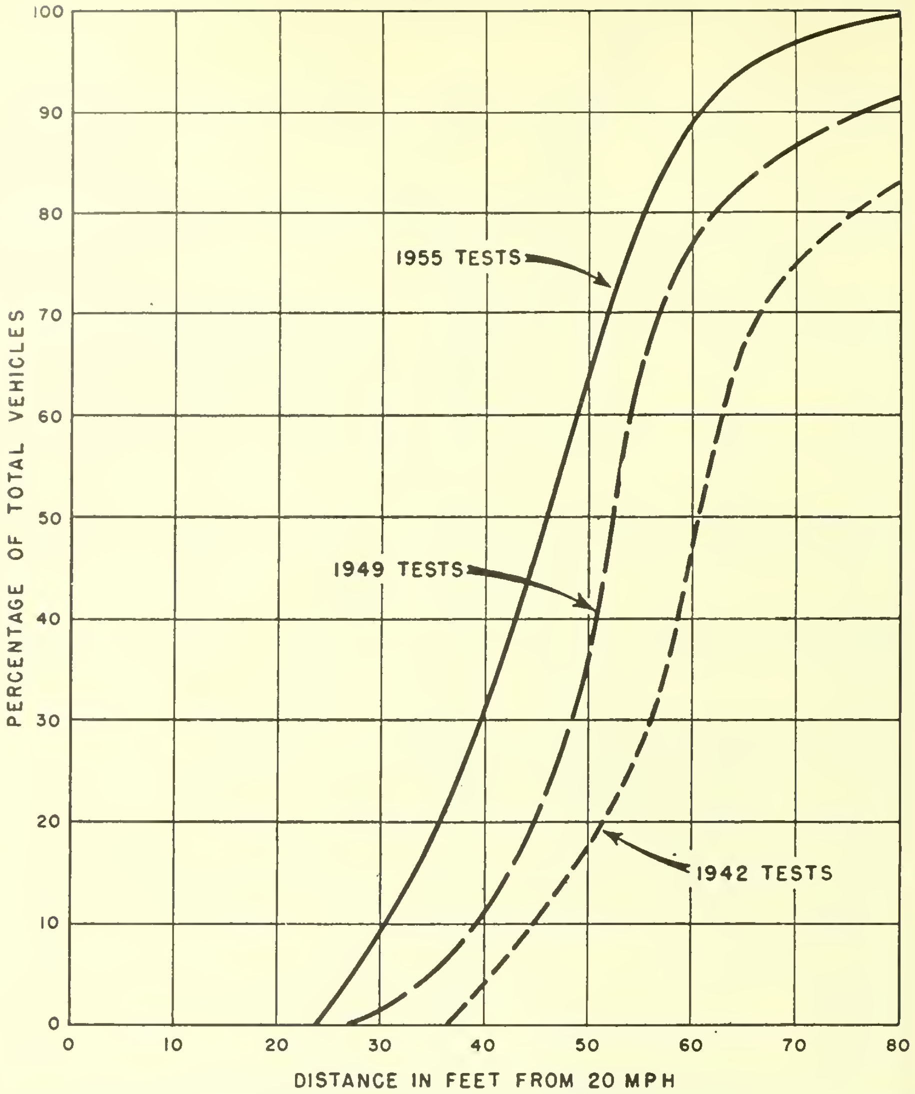

Figure 9.—Cumulative frequency distribution of minimum brake-system application and braking distances of 3-axle truck-tractors with 2-axle semitrailers.

The 3-axle single-unit trucks tested in 1955 were, on the average, 7,300 pounds heavier than those tested in 1949. This increase of approximately 35 percent in average weight was undoubtedly a contributing factor to the small improvement of 2 percent in ability to meet a 50-foot requirement for stopping ability.

Table 6.—Vehicles tested in 1955 capable of meeting the Uniform Vehicle Code requirement for deceleration

Vehicle type	Requirement for deceleration	Vehicles tested in 1955 capable of meeting requirement
	Ft./sec.^{2}	*Percent*
Passenger cars	17	99. 7
Very light 2-axle trucks	14	100. 0
Other 2-axle trucks	14	93. 9
3-axle trucks	14	85. 0
2-axle truck-tractors with 1-axle semitrailers	14	82. 7
2-axle truck-tractors with 2-axle semitrailers	14	81. 7
3-axle truck-tractors with 2-axle semitrailers	14	76. 0
Trucks with full trailers (4, 5, and 6 axles)	14	51. 2
Truck-tractors with semitrailers and full trailers (5, 6, and 7 axles)	14	69. 4

Figure 10 shows by vehicle types the improvement in braking performances which has been made in vehicles of approximately the same weight. However, it should be pointed out that the illustration does not consider vehicle capacities. Vehicles of each type were first classified into weight groups with a range of 5,000 pounds for the 2-axle trucks and 10,000 pounds for the other types. Weight groups representative of each of the seven predominant commercial vehicle types tested were selected. An average brake-system application and braking distance was then computed for each group and year of study. Because of the small sample, no results are given for the two heaviest vehicle types tested in 1942. Figure 10 emphasizes the fact that the improvement in stopping ability of the various types of vehicles between 1942 and 1949 was quite large, whereas between 1949 and 1955 there was little change for most vehicle types.

Table 8 shows that the percentage improvement, based on values indicated in figure 10, although not always identical, was similar to that determined for vehicles grouped according to capacity. The capacity groups having the greatest number of vehicles were selected

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[5] See footnote 2, p. 177.

Table 7.—Average gross vehicle weights of commercial vehicles tested in 1942, 1949, and 1955

Vehicle type [1]	Number of vehicles tested and average gross weights					
	1942		1949		1955	
	Number	Weight	Number	Weight	Number	Weight
		Pounds		*Pounds*		*Pounds*
2	1,231	13,600	367	12,200	400	11,800
3	172	24,700	46	21,100	73	28,400
2–S1	1,344	28,500	246	30,400	129	32,100
2–S2	186	35,100	98	41,500	153	40,400
3–S2	59	48,800	57	56,900	66	53,700
2–2	30	30,100	9	43,100	16	45,900
2–3	15	31,300	2	40,800	1	52,200
3–2	14	47,800	16	46,900	46	63,900
3–3	144	59,500	27	50,000	7	64,000
2–S1–2	14	39,400	50	58,100	44	59,700
2–S2–2	11	56,300	9	63,400	7	62,200
2–S2–3	6	53,100	1	94,900	2	52,000

[1] For explanation of the code used, see p. 179.

for six of the eight weight groups considered in figure 10. Where there was improvement indicated for vehicles grouped according to weight only, the further grouping according to capacity within that weight group still resulted in an improvement in 1955 when compared with 1949. Likewise, when no improvement was indicated for vehicles grouped according to weight only, no improvement was evident when further classified according to capacity. The foregoing discussion should not be interpreted as indicating that vehicle capacity has no effect on brake performance.

Gross Weight Relations

In order to demonstrate how brake performance varies with vehicle type, capacity, and brake type, it was necessary to establish a relation between weight and brake-system application and braking distance. This was accomplished by classifying vehicles of selected types and capacities into weight groups and computing the average distance for each weight group.

The relation between weight and distance is shown in figures 11 and 12 by types of vehicles, without respect to the manufacturers' ratings of capacity. Insofar as normal highway operation is concerned, brake-system application and braking distance increases with weight for a given type of vehicle; but it is the performance of the vehicles with loads normally carried that must be given primary consideration in regulatory and highway design matters.

The average brake-system application and braking distances indicated in figures 11 and 12 are summarized in table 9 for the gross weight groups having the greatest number of vehicles. These results emphasize the wide variation of stopping ability for the various types of commercial vehicles operating with their most frequently carried loads. It is significant that the range of average brake-system application and braking distances for these vehicle groups extends from 26 feet for the 2-axle trucks with a gross weight of less than 10,000 pounds to 59 feet for the heaviest truck combinations, a variation in stopping ability of 127 percent.

In figure 13 (p. 190) the performance of 2-axle trucks is compared by capacity groups. Results of tests on the heavy capacity 2-axle trucks were not included because the sample was too small. Again, it is seen that the distance required to stop increases with vehicle weight. However, figure 13 shows that the distance required to stop by the light capacity 2-axle trucks in each of the three weight groups, 10,000–14,999 pounds, 15,000–19,999 pounds, and 20,000–24,999 pounds, was greater than that required by the medium capacity vehicles in the corresponding weight groups. Further examination of the chart reveals that the stopping ability for each weight group, except the heaviest of the medium capacity 2-axle trucks, approximates very closely the stopping ability for the next lighter weight group of the light capacity vehicles. Thus, in view of the difference in rating between the two capacity groups indicated in table 3, it appears that the light and the medium capacity 2-axle trucks are rated about on a par with respect to their brake performance.

The average brake performance of light, medium, and heavy capacity 2-axle truck-tractors with 1-axle semitrailers is shown in figure 14 (p. 190). In contrast to the results shown for 2-axle trucks, the average brake-system application and braking distance for a given weight group was greater for the medium capacity vehicles than for the light capacity vehicles. The one exception involved vehicles weighing 40,000 to 49,999 pounds which, in the light capacity group, were loaded considerably beyond the manufacturer's rating.

In comparing the medium and heavy capacity vehicles, any differences in brake performance for a common weight group were generally small. The greatest difference was in the vehicles weighing 10,000 to 19,999 pounds, which were essentially empty vehicles.

In figure 15 (p. 191) the braking performance of medium and heavy 2-axle truck-tractors with 2-axle semitrailers is compared. It is significant that for a given weight group—except for the 10,000–19,999-pound group, composed of essentially empty vehicles—the

Table 8.—Improvement in stopping ability of commercial vehicles of approximately the same gross weight when grouped without respect to vehicle capacity and when grouped according to manufacturer's vehicle capacity rating

Vehicle type	Weight group	Grouped according to gross weight only				Grouped according to vehicle capacity and gross weight				
		Feet to stop from 20 m.p.h.		Improvement		Capacity group	Feet to stop from 20 m. p. h.		Improvement	
		1949	1955	Feet	Percent		1949	1955	Feet	Percent
	1,000 pounds									
2	10.0–14.9	30	31	−1	−3	Light	30	31	−1	−3
2	15.0–19.9	38	36	2	5	do	42	37	5	12
2–S1	30.0–39.9	46	41	5	11	Medium	48	44	4	8
2–S2	50.0–59.9	53	51	2	4	do	54	51	3	6
3–S2	60.0–69.9	57	47	10	18	Heavy	56	46	10	18
2–S1–2	70.0–79.9	62	62	0	0	do	62	62	0	0

Table 9.—Average brake-system application and braking distance from 20 miles per hour by vehicle type for weight groups having greatest frequency of vehicles

Vehicle type	Vehicle weight group	Number of vehicles	Average brake-system application and braking distance
	1,000 pounds		*Feet*
2-axle trucks	0–9.9	185	26
	10.0–19.9	162	32
	20.0–29.9	49	41
3-axle trucks	20.0–29.9	23	43
	30.0–39.9	22	48
2-axle truck-tractors with 1-axle semitrailers	20.0–29.9	40	37
	30.0–39.9	32	41
	40.0–49.9	35	51
2-axle truck-tractors with 2-axle semitrailers	40.0–49.9	56	46
	50.0–59.9	35	51
3-axle truck-tractors with 2-axle semitrailers	60.0–69.9	24	47
Trucks with full trailers and truck-tractors with semitrailers and full trailers	70.0–79.9	60	59

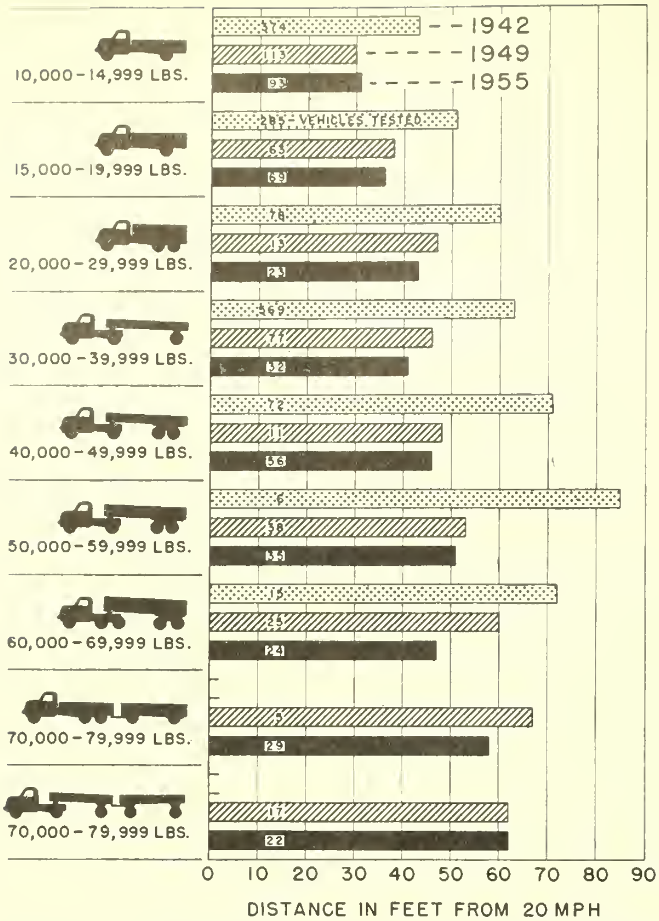

Figure 10.—Comparison of average brake-system application and braking distances observed in 1942, 1949, and 1955 for commercial vehicles loaded to commonly carried gross weights.

average distance required to stop varied with capacity group by not more than 1 foot. Thus, in considering the performance of 2-axle truck-tractors with both 1- and 2-axle semitrailers, it appears that little difference exists in the braking ability of medium and heavy 2-axle truck-tractors carrying approximately the same gross weight.

It has been demonstrated in figures 13–15 how brake-system application and braking distances of the several vehicle types vary with gross weight and with rated capacity. In the case of the 2-axle trucks, it was indicated that loads beyond the rating of the light units could be hauled in a larger capacity vehicle with an improvement in level of brake performance. This was true, also, for light capacity combination vehicles loaded considerably beyond the manufacturer's rating. However, for other 3- and 4-axle truck-tractor-semitrailer combinations vehicle capacity was not a significant factor. In fact, the overall level of performance of medium capacity 2-axle truck-tractors with either 1- or 2-axle semitrailers is not materially affected by loading somewhat beyond the rated capacity of the vehicle.

Figures 16–19 compare the braking performance of different commercial vehicle types in each of three capacity groups. Because the sample of some vehicle types in certain capacity groups was not adequate for distribution into weight groups, not every vehicle type is considered in each capacity group.

Figure 16 (p. 192) compares the stopping ability of the light capacity 2-axle trucks and 2-axle truck-tractors with 1-axle semitrailers. A comparison of identical weight groups for the two vehicle types indicates better braking performance for the combination vehicles. Because the light single-unit trucks and the power units of the light vehicle combinations are basically the same vehicles, the ability of the combination vehicles to stop in the shorter distance, when compared on an equal-weight basis, may be attributed to the added braking provided by the semitrailer.

Since the same basic vehicle is rated for a heavier gross weight when used in combination, it is more logical to compare the performance of the 2-axle trucks in one weight group with that of the 2-axle truck-tractors with 1-axle semitrailers in a heavier group. In table 3, light capacity vehicles are shown as those having a manufacturer's gross vehicle weight rating between 10,001 and 16,000 pounds for single-unit trucks and 27,000 pounds or less for combination vehicles. Therefore, it is appropriate to compare the stopping ability of the 2-axle trucks weighing 10,000–19,999 pounds with that of the 2-axle truck-tractors with 1-axle semitrailers weighing 20,000–29,999 pounds. In making such comparison, figure 16 shows slightly better performance for the single-unit vehicles. On the average, these 2-axle trucks required 33 feet to stop from 20 miles per hour, whereas the 2-axle truck-tractors with 1-axle semitrailers required 35 feet.

Figure 17 (p. 192) compares the stopping ability of medium capacity 2-axle trucks, 2-axle truck-tractors with 1-axle semitrailers, and 2-axle truck-tractors with 2-axle semitrailers. In comparing the medium capacity vehicles, it is appropriate to compare the performance of the single-unit vehicles weighing 20,000–29,999 pounds with that of the combination vehicles weighing 40,000–49,999 pounds, since these weight groups bracket the upper limits of the respective medium capacity ratings given in table 3. In comparing the vehicles on this basis, figure 17 shows the following average brake-system application and braking distances for stops made from 20 miles per hour: 2-axle trucks, 42-feet; 2-axle truck-tractors with 1-axle semitrailers, 52 feet; and 2-axle truck-tractors with 2-axle semitrailers, 46 feet. Again, as in the case of the light capacity vehicles, the 2-axle trucks showed the better performance.

The somewhat poorer performance of combination vehicles may largely be attributed to the inherent characteristics of the braking systems. A certain amount of additional time is required to transmit power to, and to build up force in the trailer brakes. During this time the vehicle combination is traveling faster than at any other time during the stop. Therefore, for a given increment of time, it travels farther and the brake-system application and braking distance is bound to be lengthened somewhat. In the best performing vehicles the application and brake force buildup time is held to a minimum.

In comparing the performance of the two medium capacity truck-tractor-semitrailer types considered in figure 17, it is seen that the average brake-system application and braking distance of each weight group of the combinations with single-axle semitrailers was approximately equal to that of the next heavier weight group of the combinations with tandem-axle semitrailers. For example, truck-tractors with single-axle semitrailers weighing 20,000–29,999 pounds and the combinations with tandem-axle semitrailers weighing 30,000–39,999 pounds, on the average, required 38 feet to stop. Figure 18 (p. 193) indicates the same general relation for the heavy capacity vehicles of these two types. Thus, for vehicles within the same capacity group, it appears that the brake performance of a 2-axle truck-tractor with 2-axle semitrailer of a given gross weight is generally on a par with the performance of a 2-axle truck-tractor with 1-axle semitrailer of about 10,000 pounds less weight.

In figure 19 (p. 193) the stopping ability of heavy capacity 3-axle truck-tractors with 2-axle semitrailers is compared with that of heavy capacity truck and trailer and truck-tractor-semitrailer and full-trailer combinations. The illustration shows that in each weight group the 3-axle truck-tractors with 2-axle semitrailers required less distance to stop than the other vehicle combinations in the corresponding weight group.

The results shown in figure 19 and those shown in figures 10 and 17 indicate that brake-

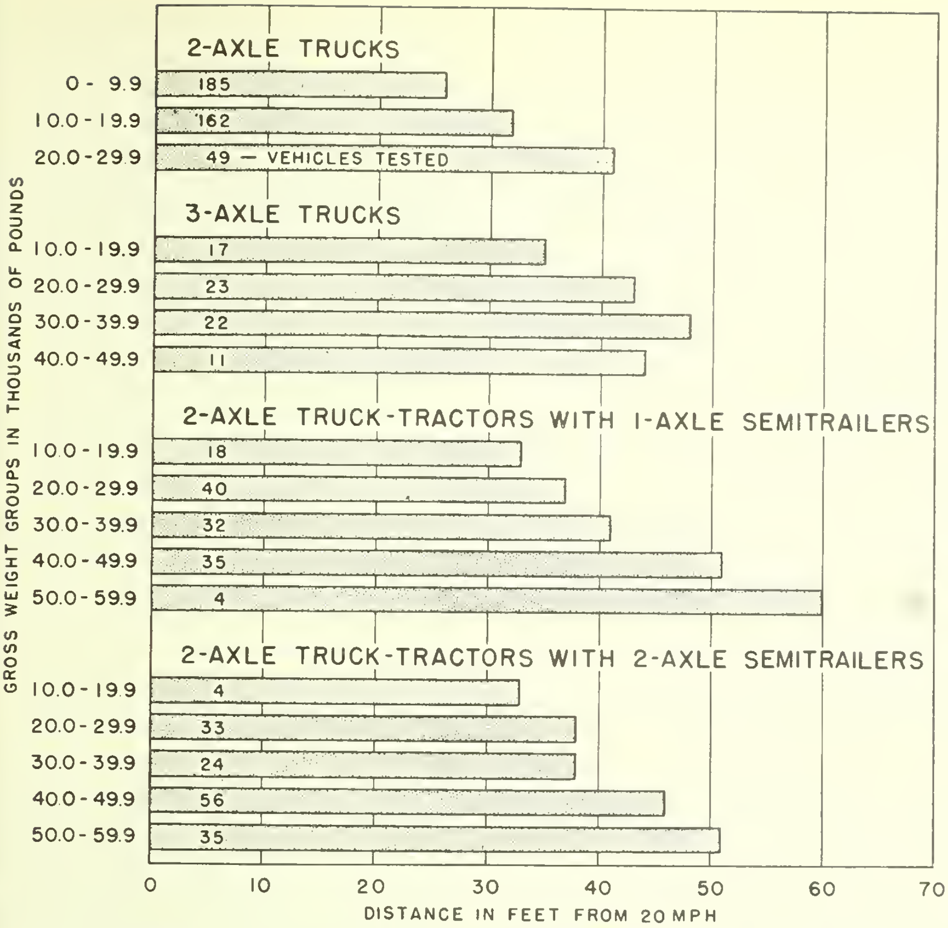

Figure 11.—Variation of average brake-system application and braking distances with gross weights for single-unit trucks and truck-tractor-semitrailer combinations.

system application and braking distance, for vehicles within a given capacity group, increases with the number of units in combination. The additional time required in application and brake force buildup, as mentioned in the previous discussion of figure 17, accounts for the greater distance in stopping combination vehicles. When an additional unit is included in a combination, additional time is required to transmit power to, and to build up full braking force in some of the brakes of the combination. This is due primarily to the longer lines and added connections involved in triggering the power to operate the brakes.

Brake Performance Summarized

In order to define the practical levels of brake performance for vehicles currently using the highways, the average, 15-, and 85-percentile values of brake-system application and braking distance observed for vehicles as commonly loaded are summarized in table 10 by vehicle type, capacity, and weight range. Most of the previous analyses have been concerned only with the average performance level for a given group of vehicles. In this table, the 15- and 85-percentile values are included to indicate the spread of data about the average.

The 15-percentile values represent the performance of the top 15 percent of the vehicles in a group; and the 85-percentile values, the performance attained by 85 percent of the vehicles (and not attained by the poorest 15 percent). The 85-percentile value may be useful for highway design purposes, whereas the 15-percentile value gives some idea of what can be expected in the future.

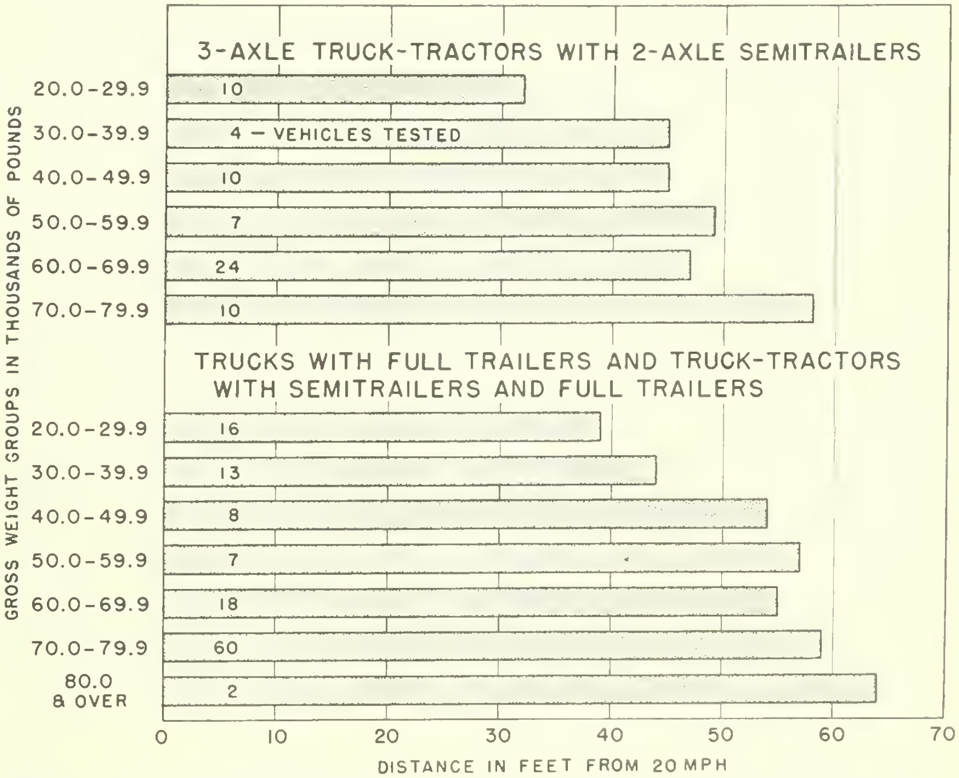

Figure 12.—Variation of average brake-system application and braking distances with gross weights for the larger truck combinations.

A significant point to be made from this summary is the need for a concentrated effort to bring the brake performance of truck-tractors with semitrailers and full trailers, trucks with full trailers, and the heaviest groups of truck-tractors with semitrailers into line with the performance of the other groups of truck-tractors with semitrailers. The results should also prove beneficial in testing the practicability of minimum regulatory requirements and in developing highway design factors. The 15-percentile values show definitely that considerable improvement is possible for each type of vehicle.

Axle Weight Relations

In the previous analyses, brake-system application and braking distances have been related either directly or indirectly to gross vehicle weight. The following discussion considers the relation existing between axle load and brake-system application and braking distance for 2-axle trucks and 2-axle truck-tractors with 1-axle semitrailers. It was not possible to consider other vehicle types since the number of vehicles of a given type was either too few, or the typical axle-load distribution for a given type of vehicle was too varied.

In order to establish the relation, vehicles were first classified by axle-weight groups with a range of 4,000 pounds. Only the principal load carrying axles were considered; the weight of the front axle was disregarded. If both axles of a truck-tractor-semitrailer combination did not fall in the same weight group, that particular vehicle was not considered. Following the classification, an average brake-system application and braking

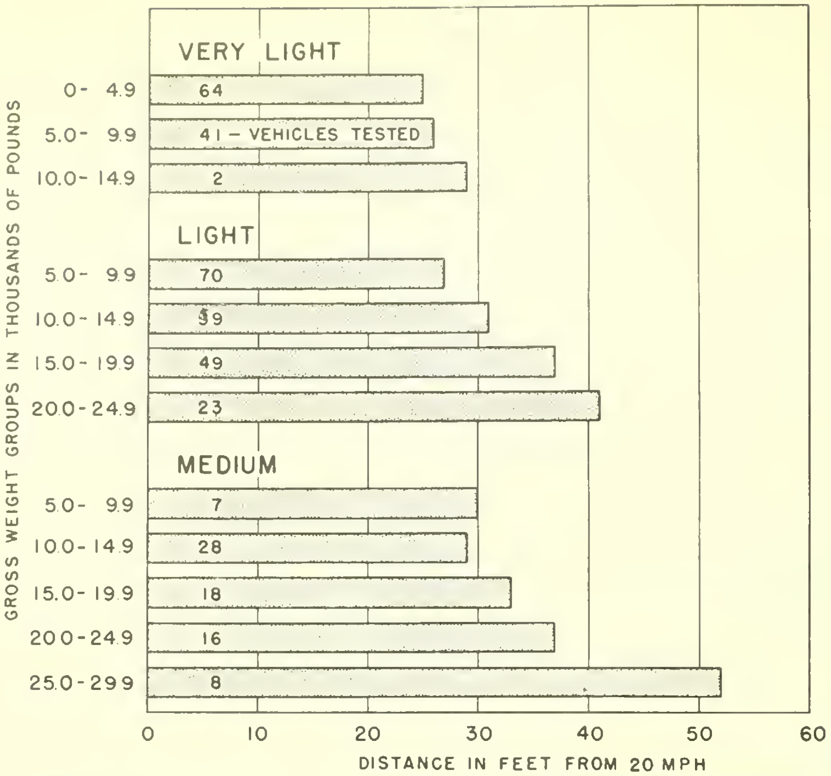

Figure 13.—Variation of average brake-system application and braking distances with gross weights by capacity groups for 2-axle trucks.

distance was computed for each axle-weight group.

The results are shown in figure 20 (p. 194) for vehicles tested in 1955 and 1949. So far as the general traffic is concerned, average brake-system application and braking distance for both years definitely increased with an increase in axle load. This would be expected, to some degree, since many vehicles were undoubtedly tested with either partial loads or loads in excess of the brake design. Except for the vehicles in the 14,000–17,999-pound axle-weight group, there was little change in the relation between brake performance and axle load from 1949 to 1955. The improvement in braking performance for vehicles in the 14,000–17,999-pound group amounted to 7 feet for the 2-axle trucks, and 4 feet for the 2-axle truck-tractors with 1-axle semitrailers.

In viewing the results presented in figure 20, it should not be construed that heavy axle loads cannot be adequately braked. The report, published in 1954, pointed out that a few of the vehicles tested in that study showed that axle loads commonly carried could be braked to the degree of wheel locking on dry concrete pavement with a relatively high coefficient of friction. One of several examples cited was a 2-axle truck-tractor with 1-axle semitrailer carrying 22,900 pounds on the tractor drive-axle and 22,800 pounds on the trailer axle. After adjusting the brakes just prior to testing, this vehicle stopped in 32 feet from 20 miles per hour. Other heavily loaded vehicles tested in that study showed equally good brake performance.

A few of the vehicles tested in 1955 add further proof that axle loads now commonly carried can be adequately braked. It should be remembered that all of the vehicles tested in the present study were selected at random from the general traffic, and in no instance was special maintenance performed for the tests. There were four examples of heavily loaded 2-axle truck-tractors with 1-axle semitrailers. The weights of the drive axle and trailer axle, along with the brake-system application and braking distance for each of the four respective vehicles, were as follows: 21,000 and 22,800 pounds, 36 feet; 19,500 and 23,800 pounds, 36 feet; 22,500 and 22,100 pounds, 38 feet; and 20,400 and 20,400 pounds, 38 feet. Results of the controlled tests reported in 1954 indicate that these vehicles could have done even better if the brakes had been adjusted prior to the tests.

Regardless of what can be done, the fact remains that commercial vehicles from the general traffic—at least the two types considered in figure 20, and very likely all types—were not provided with braking ability in proportion to the loads carried on an axle. In view of the performance possible with current designs of braking systems, and in view of the performance of a few of the vehicles selected at random from the general traffic, the range of about 25 feet between the lightest and heaviest axle-load groups seems excessive. Some of the increase of brake-system application and braking distance with axle load might be attributed to longer application and buildup time, since the frequency of use of power-brake systems increases with axle load. However, the magnitude of the difference between what can be done and what is generally being done suggests that insufficient attention is given to providing adequate braking force for the axle loads carried.

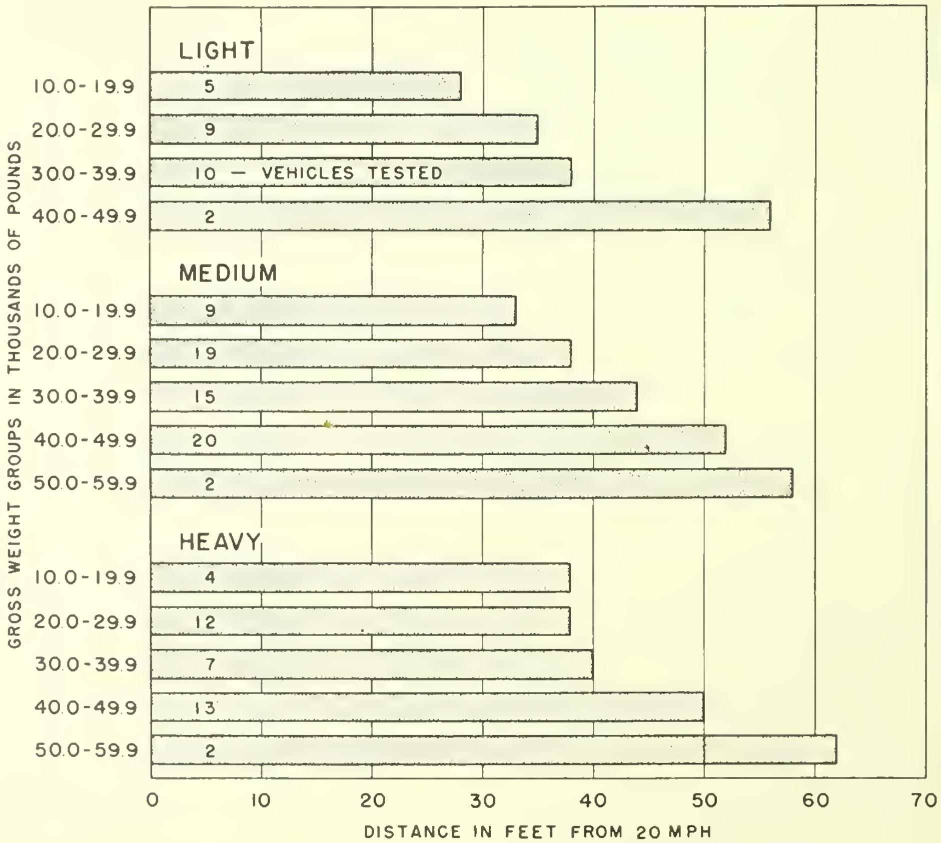

Figure 14.—Variation of average brake-system application and braking distances with gross weights by capacity groups for 2-axle truck-tractors with 1-axle semitrailers.

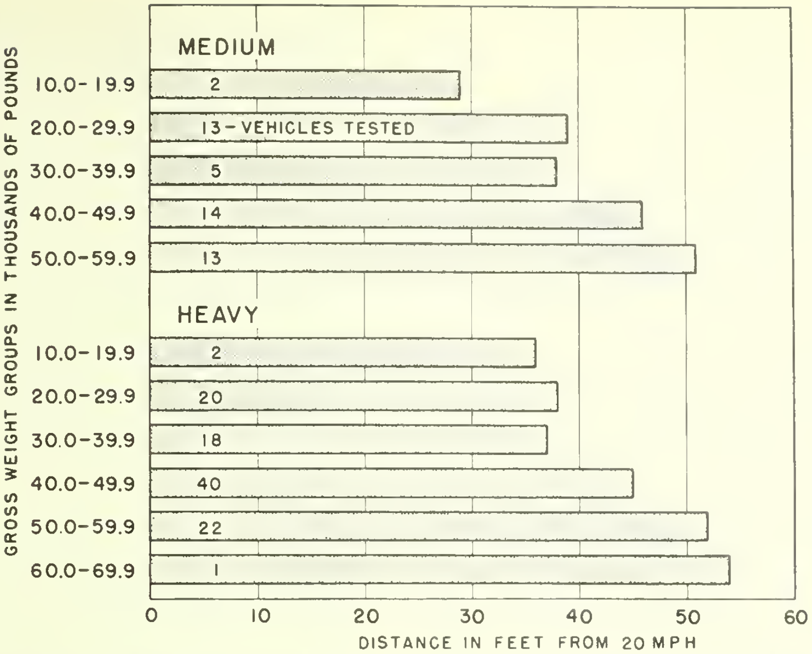

Figure 15.—Variation of average brake-system application and braking distances with gross weights by capacity groups for 2-axle truck-tractors with 2-axle semitrailers.

The existing condition, as revealed in figure 20, may be attributed to loading beyond the axle loads used in designing the brake components, inadequate maintenance of equipment, and obsolete design of the overall braking system. The operators have the responsibility of loading the axles in keeping with the braking force available, of properly maintaining the braking system, and of modernizing old equipment still being used on the highways. There is also a joint responsibility of both operators and manufacturers in the selection of and the provision for adequate braking equipment.

Brake-Type Relations

From the classification of vehicles made in table 2, it is apparent that comparisons of brake performance by brake type were possible for only the passenger cars, light 2-axle trucks, and medium 2-axle truck-tractors with 1-axle semitrailers. It was not possible to compare the stopping ability by brake type for other groups of vehicles because of the limited sample and the nature of its distribution by vehicle type, capacity group, and brake type.

For passenger cars, the stopping ability of vehicles equipped with conventional hydraulic brakes was compared with the stopping ability of vehicles equipped with vacuum-booster hydraulic brakes (commonly called power brakes). Average brake-system application and braking distance was computed for each of the two groups of passenger cars. Since the cars equipped with power brakes were predominantly 1953–56 models, only those models were included in the computations. The results showed that the average brake-system application and braking distance for passenger cars with hydraulic brakes was 20.5 feet, and with power (vacuum-booster hydraulic) brakes, 21.5 feet.

The relation established between brake type and brake-system application and braking distance for commercial vehicles of the same type, capacity, and approximate weight is shown in tables 11 and 12. The average performance of light 2-axle trucks with hydraulic and with vacuum-booster hydraulic brakes is compared in table 11 for three weight groups. The results indicate slightly better performance for the vehicles with vacuum-booster hydraulic brakes in the 10,000–14,999-pound group, and identical performance for the two brake types in the other two weight groups.

Results of the 1949 tests indicated that a power booster on the brakes of light trucks in the 10,000–14,999-pound group was desirable, and that light vehicles weighing 15,000 pounds and over should be equipped with a booster. The present study tends to indicate that a booster is not beneficial (except for reducing driver fatigue) on the brakes of present light 2-axle trucks if they are not loaded beyond the manufacturer's gross weight rating.

Table 12 shows the average brake performance for each of three gross weight groups of medium capacity 2-axle truck-tractors with 1-axle semitrailers equipped with two different types of brakes. Average brake-system application and braking distances from 20 miles per hour are shown for vehicle combinations with vacuum-booster hydraulic brakes on the truck-tractor and vacuum-mechanical brakes on the semitrailer, and for combinations with air-mechanical brakes on both units. The table indicates that vehicles with air-mechanical brakes were capable of stopping in 2 to 5 feet shorter distance than those equipped with vacuum-power systems.

Table 10.—Summary of brake-system application and braking distances from 20 miles per hour for commercial vehicles with gross weights commonly carried on the highway

Vehicle type	Capacity group	Weight group	Number of vehicles	Brake-system application and braking distance		
				Average	15-percentile level	85-percentile level
		1,000 pounds		*Feet*	*Feet*	*Feet*
2	Very light	0 – 4. 9	61	25	20	30
	do	5. 0– 9. 9	33	26	22	30
	Light	10. 0–14. 9	60	31	24	37
	do	15. 0–19. 9	49	37	28	48
	do	20. 0–24. 9	23	41	29	48
	Medium	15. 0–19. 9	18	33	30	36
	do	20. 0–24. 9	16	37	31	43
	do	25. 0–29. 9	8	52	37	63
	Heavy	20. 0–29. 9	2	38	------	------
	do	30. 0–34. 9	3	44	------	------
3	Light	20. 0–29. 9	6	42	------	------
	Medium	20. 0–29. 9	6	45	------	------
	do	30. 0–39. 9	11	48	37	58
	Heavy	20. 0–29. 9	11	42	29	49
	do	30. 0–39. 9	10	47	34	53
	do	40. 0–49. 9	6	43	------	------
2-S1	Light	20. 0–29. 9	9	35	27	37
	do	30. 0–39. 9	10	38	29	45
	Medium	30. 0–39. 9	15	44	36	50
	do	40. 0–49. 9	20	52	38	62
	Heavy	30. 0–39. 9	7	40	33	45
	do	40. 0–49. 9	13	50	42	53
	do	50. 0–59. 9	2	62	------	------
2-S2	Medium	40. 0–49. 9	14	46	36	54
	do	50. 0–59. 9	13	51	39	58
	Heavy	40. 0–49. 9	40	45	39	51
	do	50. 0–59. 9	22	52	41	59
3-S2	Heavy	60. 0–69. 9	20	46	37	54
	do	70. 0–74. 9	10	58	50	63
2-2	Heavy	60. 0–64. 9	5	56	------	------
3-2	do	70. 0–79. 9	29	58	45	67
3-3	do	70. 0–79. 9	5	65	------	------
2-S1-2	do	70. 0–79. 9	20	62	48	74
2-S2-2	do	75. 0–84. 9	4	59	------	------

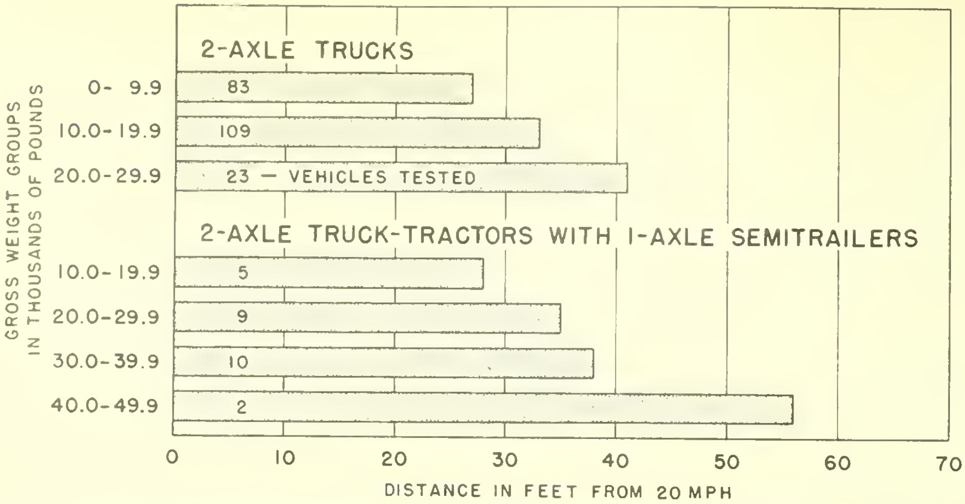

Figure 16.—Variation of average brake-system application and braking distances with gross weights for light capacity single-unit trucks and truck combinations.

Table 12.—Relation between brake type and brake-system application and braking distance for medium capacity 2-axle truck-tractors with 1-axle semitrailers

Gross combination weight group (pounds)	Number of vehicles		Average brake-system application and braking distance from 20 m. p. h.	
	VBH-VM brakes [1]	AM-AM brakes [1]	VBH-VM brakes [1]	AM-AM brakes [1]
			Feet	*Feet*
20,000-29,999	7	8	40	35
30,000-39,999	8	6	46	42
40,000-49,999	4	11	51	49

[1] VBH=vacuum-booster hydraulic, VM = vacuum-mechanical, and AM=air-mechanical.

The 1949 tests showed a similar relation in performance of the two brake types for vehicles selected from the general traffic. However, controlled tests conducted during the earlier study showed that the 2-axle truck-tractors with 1-axle semitrailers equipped with vacuum-booster hydraulic brakes on the tractor and vacuum-mechanical brakes on the trailer could perform as well as similar vehicles equipped with air-mechanical brakes. This was true for vehicles weighing up to 40,000 pounds, provided proper components of the braking systems were selected and used, and provided the braking systems were properly maintained.

Brake Performance Compared by Regions

The previous analyses have been concerned with average brake-system application and braking distances for all vehicles tested in the three States. Each State actually represents a region since the normal sample of out-of-State vehicles was included. Table 13 shows, by States, the average brake performance and the average gross weight of each of the most common vehicle types tested in 1955. It is interesting to note that 10 of the 11 vehicle types considered showed better average stopping ability in Michigan than in either Maryland or California. For some vehicle types the better showing may be partly attributed to differences in average gross weights. However, the average weight was heavier in Michigan for 4 of the 10 types which showed the best performance in that State. The average weight of the one type of vehicle with the poorest performance in Michigan, the 2-axle truck-tractor with 2-axle semitrailer and 2-axle trailer, was 20,600 pounds heavier than the same type tested in California.

In general, the average level of performance was better in Maryland than California. Of the 7 vehicle types tested in both States, 5 had a better average braking performance in Maryland than in California. The average weight was heavier in Maryland for 3 of the 5 vehicle types. Passenger cars showed about 10 percent better braking performance in California, and the very light 2-axle trucks required the same distance to stop in both States. However, the average weight of each of these two vehicle types was slightly heavier in Maryland.

Since the improvement in stopping ability of the 3-axle truck-tractors with 2-axle semitrailers has previously been emphasized, it is of particular interest to investigate the performance of this vehicle type in the individual States. Because only two vehicles of this type were tested in Maryland, comparisons are confined to vehicles tested in Michigan and in California.

Figure 21 shows four cumulative frequency distribution curves for brake-system application and braking distances of 3-axle truck-tractors with 2-axle semitrailers. Two curves represent the vehicles tested in Michigan and California in 1955, and the other two represent the total samples of this vehicle type tested in

Table 11.—Relation between brake type and brake-system application and braking distance for light 2-axle trucks

Gross vehicle weight group (pounds)	Number of vehicles		Average brake-system application and braking distance from 20 m. p. h.	
	Hydraulic brakes	VBH brakes [1]	Hydraulic brakes	VBH brakes [1]
			Feet	*Feet*
5,000-9,999	10	59	26	26
10,000-14,999	9	50	33	31
15,000-19,999	3	44	38	38
20,000-24,999	..	22	..	39

[1] VBH=vacuum-booster hydraulic.

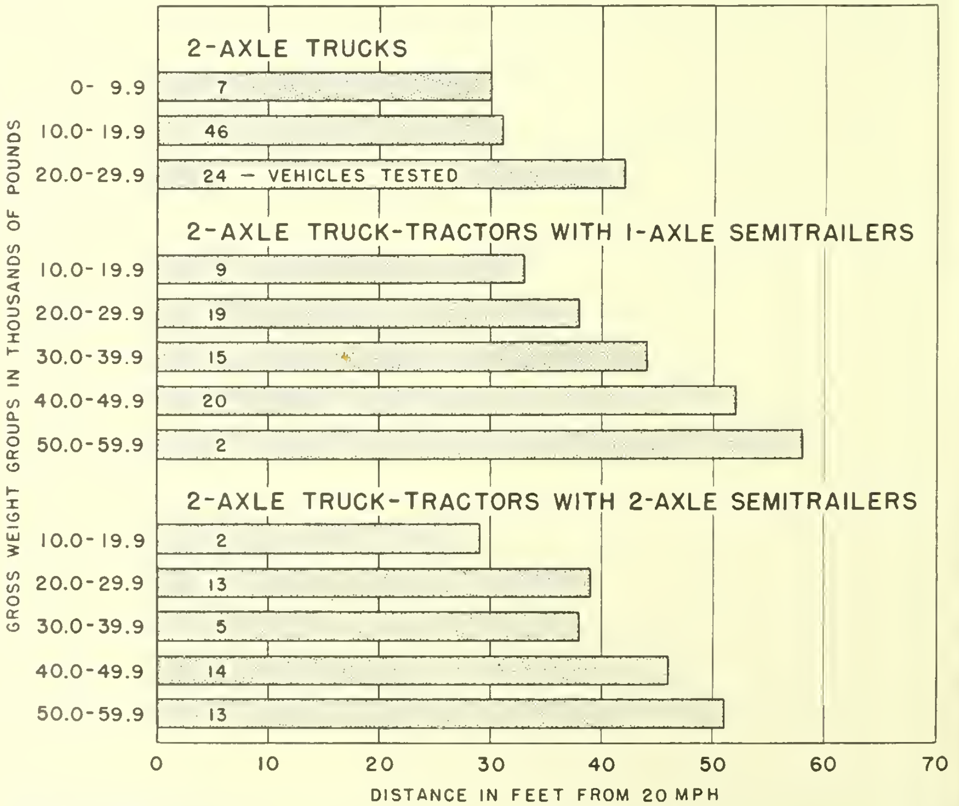

Figure 17.—Variation of average brake-system application and braking distances with gross weights for medium capacity single-unit trucks and truck combinations.

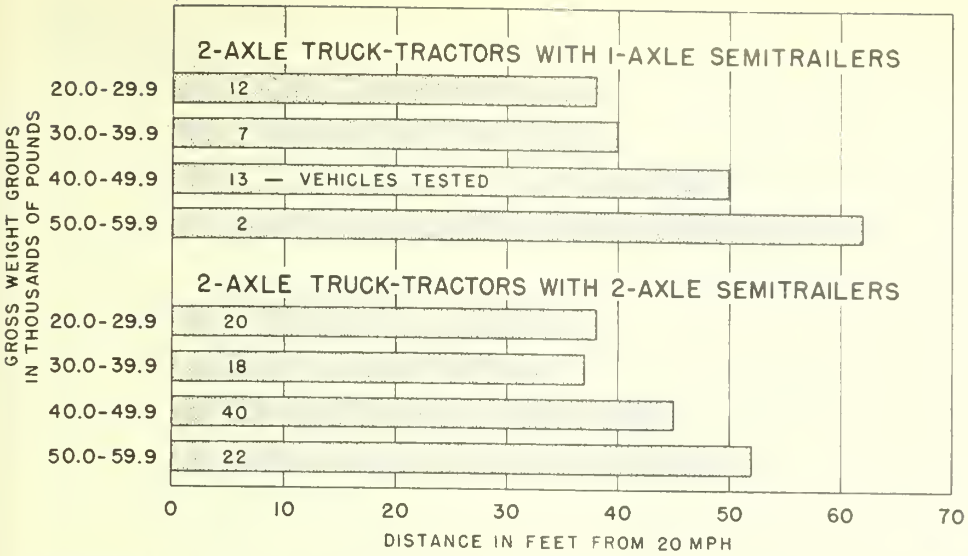

Figure 18.—Variation of average brake-system application and braking distances with gross weights for heavy capacity truck combinations.

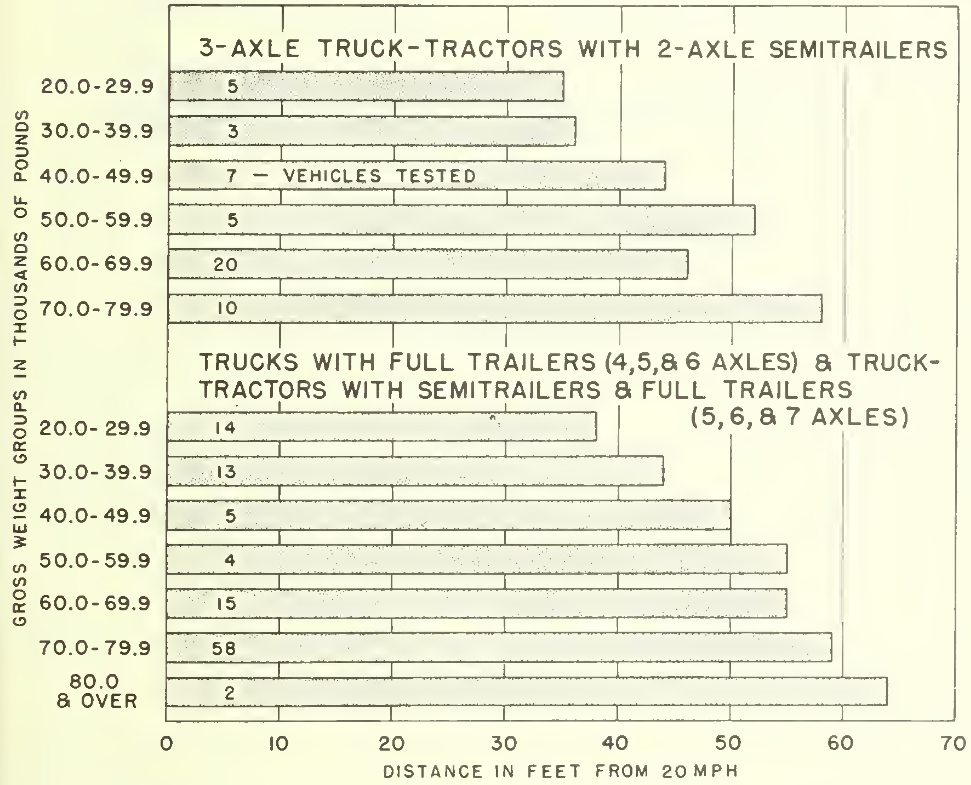

Figure 19.—Variation of average brake-system application and braking distances with gross weights for the larger heavy capacity truck combinations.

1955 and 1949. The 1949 curve can be considered as essentially representing the performance of vehicles tested in California, as only 2 of the 57 vehicles it represents were tested elsewhere. The wide variation in performance of similar vehicles tested in the two States is of particular interest. The graph indicates that only small improvements in the performance of these vehicles were made in California between 1949 and 1955. The greater part of the improvement for the total sample was due to the performance of vehicles tested in Michigan.

Table 14.—Comparison of brake-system application and braking distance by weight groups for 3-axle truck-tractors with 2-axle semitrailers tested in California and Michigan

Gross combination weight group (pounds)	Number of vehicles		Average brake-system application and braking distance from 20 m. p. h.	
	California	Michigan	California	Michigan
			Feet	*Feet*
20,000–29,999	1	9	43	31
30,000–39,999	4	- -	45	
40,000–49,999	6	3	49	37
50,000–59,999	6	1	54	55
60,000–69,999	9	15	52	44
70,000–74,999	9	1	59	51
All groups	35	29	52	40

It is pointed out later that differences in terrain may contribute to differences in performance of vehicles tested in different States. However, for the 3-axle truck-tractors with 2-axle semitrailers some of the difference should be attributed to another factor. Table 1 shows that in 1949 only two vehicle combinations of this type were tested in Michigan. Since 1949, this vehicle type has been gaining in popularity in the Midwest. The data recorded in 1955 show that on the average the age of the power units of these vehicles tested in California was 5 years, and in Michigan, 2½ years. Thus, it is probable that the 3-axle truck-tractors with 2-axle semitrailers tested in Michigan were generally equipped with more modern braking equipment.

Since table 13 showed that the average weight of these vehicle combinations was 8,100 pounds less in Michigan, it might be construed that the difference in weight was a major contributing factor to the difference in performance. Table 14 compares by weight groups the average brake-system application and braking distances for 3-axle truck-tractors with 2-axle semitrailers of approximately the same weight tested in each of the two States. It is significant that in general the vehicles of this type tested in Michigan were able to stop in 8 to 12 feet shorter distance than those of approximately equal weight tested in California. The results shown in table 14

Table 13.—Average minimum brake-system application and braking distance and average gross weight by States for various vehicle types

Vehicle type	Brake-system application and braking distance			Gross weight		
	Maryland	Michigan	California	Maryland	Michigan	California
	Feet	*Feet*	*Feet*	*Pounds*	*Pounds*	*Pounds*
Passenger cars	23	19	21	4, 050	4, 000	3, 850
Single-unit trucks:						
2-axle (very light)	26	23	26	5, 250	5, 600	4, 950
2-axle (other than very light)	35	30	36	15, 700	12, 800	14, 500
3-axle	43	37	48	26, 600	30, 600	27, 500
Truck-tractors with semitrailers:						
2–S1	42	41	45	33, 900	31, 300	28, 000
2–S2	44	43	46	40, 800	40, 200	38, 900
3–S2	42	40	52	50, 900	49, 300	57, 400
Trucks with full trailers:						
2–2	--------	48	55	--------	48, 100	43, 200
3–2	--------	48	57	--------	47, 800	70, 200
Truck-tractors with semitrailers and full trailers:						
2–S1–2	--------	54	56	--------	65, 100	58, 300
2–S2–2	--------	57	46	--------	70, 600	50, 000

substantiate the relative performances indicated in figure 21.

The relative performance by States of the vehicles considered in table 13 undoubtedly reflects the type of terrain they normally traverse. Most of the vehicles tested in Michigan usually travel over level to rolling terrain; whereas in Maryland, hilly and mountainous country is encountered; and in California, mountainous terrain with long sustained grades is not uncommon. Where long or steep grades are found, wear of the brakes is generally more severe than in flat country, and more frequent brake adjustments are needed. The differences in brake performance by regions emphasize the need for more frequent brake maintenance for vehicles which travel over hilly or mountainous terrain.

Incidental Observations

A considerable amount of data which may be termed incidental to the main purposes of the study was obtained in connection with the brake tests. Much of these data are interesting and useful for reference purposes, and in some cases have a bearing on the results just reported.

Pedal reserve

Pedal reserve is the distance in inches between the floorboard and the back of the brake pedal at the end of an emergency stop. Except for vehicles equipped with certain power-brake systems which upon full application the pedal rests on the floorboard, zero pedal reserve is an indication of need for brake maintenance. A comparison of the percentages of vehicles having zero pedal reserve in the 1942, 1949, and 1955 studies is shown in table 15. The decrease in percentages of vehicles with zero pedal reserve in 1949, and again in 1955, probably reflects improvements in both maintenance and design of the braking systems. The magnitude of the reductions is in keeping with improvements in stopping ability which were previously discussed.

Swerving

The term "swerving" describes an uncontrollable lateral movement of the vehicle when the brakes are applied. In severe cases the vehicle may enter the opposing traffic lane or leave the road. If this condition exists in stops made from 20 miles per hour, experience has shown that it is usually accentuated in stops from higher speeds. This is one very important reason why tests on the highway should not be conducted at a speed greater than 20 miles per hour. Some vehicles of each type swerved to some extent. The passenger cars and the very light 2-axle trucks were the worst offenders. A comparison of percentages of vehicles that swerved in the 1942, 1949, and 1955 studies is given in table 15. The condition of swerving appears to have improved somewhat since 1949, but there is need for much more improvement, particularly in passenger cars and single-unit trucks.

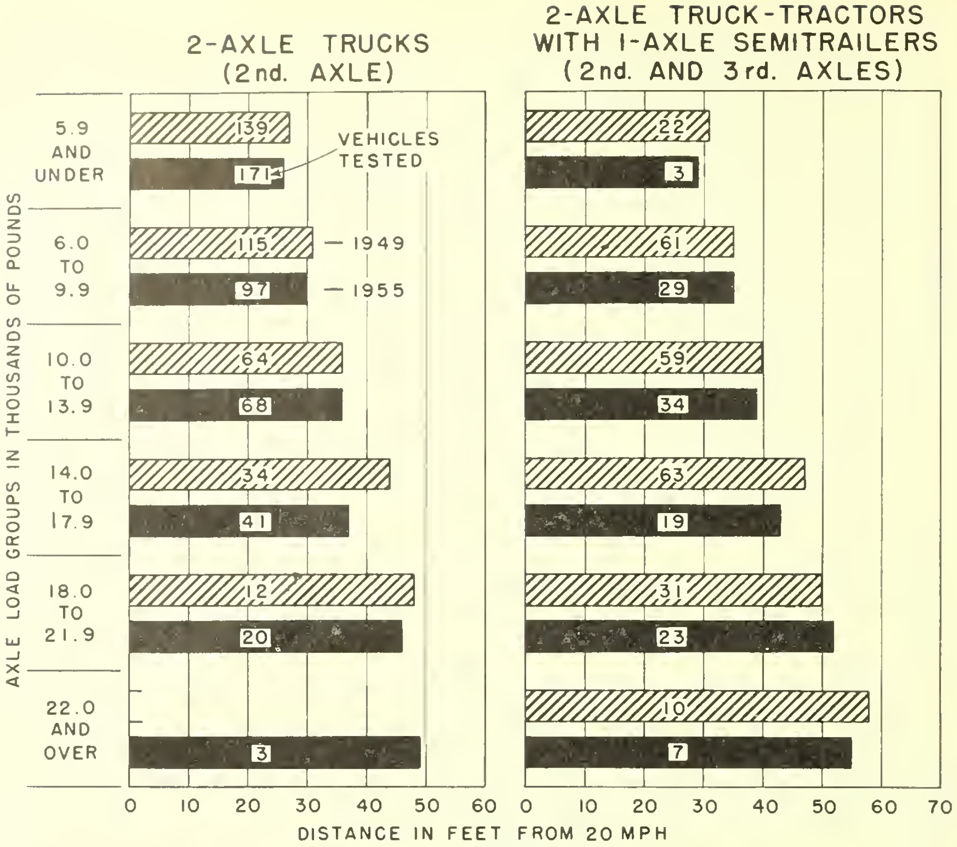

Figure 20.—Relation between axle loads and average brake-system application and braking distances for single-unit trucks and truck combinations.

Wheel slides

The sliding of wheels on a vehicle making a stop indicates that more than adequate braking force is being applied, at least for some part of the stop, with respect to the wheel loads and the frictional characteristics of the particular pavement and tire. The ideal condition, which is difficult to attain because of varying conditions of pavement surface, tires, and brake lining, is to apply just enough braking force to bring the wheel to the point of impending skidding. The fact that a wheel does not slide does not necessarily mean that more braking force is needed. However, the number of vehicles of a sample that slid wheels, when considered in connection with the general levels of brake performance, does give a clue as to whether more braking force could be used.

The percentage of vehicles of various types that slid one or more wheels on any axle is

Table 15.—A comparison of the percentages of vehicles with zero pedal reserve after an emergency stop, and the percentages of vehicles swerving following brake application in the 1942, 1949, and 1955 studies

Vehicle type	Percentage of vehicles with zero pedal reserve			Percentage of vehicles swerving		
	1942	1949	1955	1942	1949	1955
Passenger cars	41	7	3	20	34	28
Very light 2-axle trucks	(1)	(1)	2	(1)	(1)	29
All 2- and 3-axle trucks	56	10	4	16	18	14
2-axle truck-tractors with 1-axle semitrailers	75	7	2	12	12	5
Combination vehicles with 4 or more axles	(2)	(2)	(2)	6	8	4

[1] Percentages not determined. [2] Not applicable.

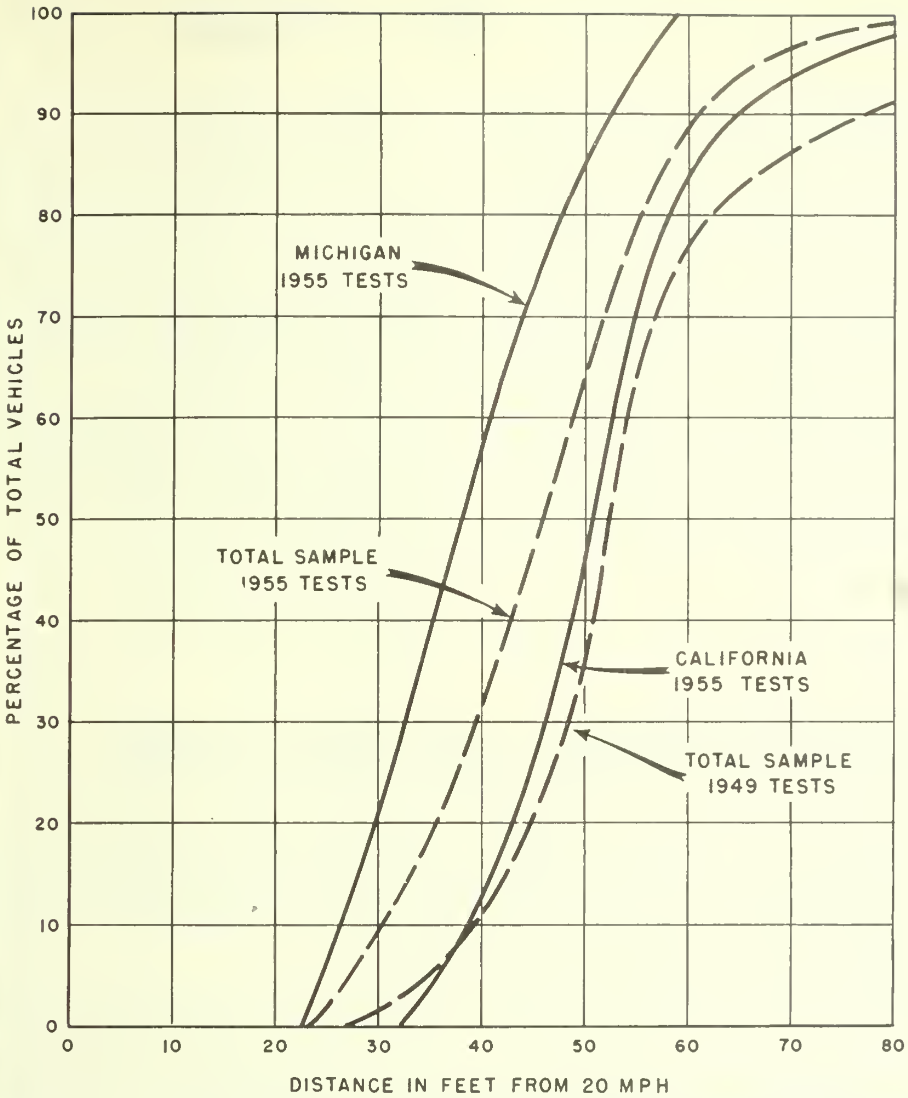

Figure 21.—Cumulative frequency distribution of minimum brake-system application and braking distances for 3-axle truck-tractors with 2-axle semitrailers tested in Michigan and California in 1955.

Table 16.—Percentage of vehicles sliding one or more wheels

Vehicle type	Percentage of vehicles sliding wheels on—							
	Front axle	Second axle	Third axle	Fourth axle	Fifth axle	Sixth axle	Seventh axle	One or more axles
Passenger cars	54	74	----	----	----	----	----	85
Very light 2-axle trucks	18	74	----	----	----	----	----	76
Other 2-axle trucks	6	48	----	----	----	----	----	51
3-axle trucks	0	45	48	----	----	----	----	51
2-axle truck-tractors with 1-axle semitrailers	0	26	31	----	----	----	----	40
2-axle truck-tractors with 2-axle semitrailers	1	17	37	45	----	----	----	53
3-axle truck-tractors with 2-axle semitrailers	0	17	18	27	38	----	----	50
Trucks with full trailers (4, 5, and 6 axles)	0	9	7	10	19	3	----	29
Truck-tractors with semitrailers and full trailers (5, 6, and 7 axles)	0	11	33	22	48	4	4	61

shown in table 16. A relatively small percentage of the commercial vehicles, especially the combination vehicles, slid wheels as compared with passenger cars. Percentages of empty trucks and combinations sliding wheels were as follows: 40 percent of the very light 2-axle trucks, 37 percent of the other single-unit vehicles, 31 percent of the truck-tractors with semitrailers, and 31 percent of the other vehicle combinations. Many of the remaining vehicles with sliding wheels were only partly loaded.

Thirty-five percent of the passenger cars, 8 percent of the very light 2-axle trucks, and 2 percent of the other 2-axle trucks slid all wheels. In no instance did this happen to 3-axle trucks or vehicle combinations.

From the foregoing discussion of the vehicles with sliding wheels, and from the levels of performance described earlier in this article, it is reasonable to assume that loaded combination vehicles now operating in the general traffic could use additional braking force. The deficiency in braking effort, if that is the case, could be attributed either to loads greater than those for which the brakes were designed or to inadequate brake-system maintenance.

Miscellaneous vehicle combinations

Included in the 1955 study, as shown in table 1, were three passenger-car driveaway-towaway combinations and five housetrailer factory-towaway combinations. Although the performance of such a small sample cannot be considered representative, it does provide some indication of the stopping ability of these combinations. The three passenger-car towaway combinations required 39, 46, and 67 feet to stop. In the case of the combination which stopped in 39 feet, the brakes on the towed car, as well as those on the towing car, were applied in stopping. Both of these cars were new. The other two towaway combinations were secondhand cars, and the brakes were operative on only the towing vehicle.

The power units of the five housetrailer factory-towaway combinations were very light trucks with manufacturers' gross weight ratings varying from 5,800 to 9,500 pounds for use as single-unit vehicles. These combinations were equipped with hydraulic brakes on the power units and electric brakes on the trailers. The average gross weight of the five combinations was 12,800 pounds, and their average brake-system application and braking distance was 36 feet. Additional housetrailer combinations were stopped, but the drivers requested that they be excused from the test because of the possibility of damaging refrigerators or other equipment inside of the trailers.

Construction Specifications for Federal Highway Projects

The Bureau of Public Roads has recently published a 363-page book entitled *Standard specifications for construction of roads and bridges on Federal highway projects.* As indicated by the title, the book is intended primarily for use in the construction of Federal road and bridge projects under the direct supervision of the Bureau. Such projects include work for which Public Roads receives direct appropriations, as for major highways through National forests, and work performed by Public Roads for other Federal agencies.

The book, which for simplified reference may be cited as FP–57, supersedes a previous publication, *Specifications for construction of roads and bridges in National forests and National parks* (FP–41), issued in 1941.

The new book contains up-to-date specifications for those items of work and materials and construction methods that are generally applicable to direct Federal highway contracts. They are considered to be good specifications, which will result in highway work of high quality. These specifications are not required or intended to be used in Federal-aid highway work performed by the States with funds administered by the Bureau of Public Roads since, as prescribed in the basic Federal-aid highway legislation, each State prepares its own specifications for Federal-aid highway construction, subject to approval by Public Roads. However, these specifications will undoubtedly be of interest to specifications writers in particular and to most engineers in highway design and construction, as well as to engineering students.

During the preparation of the new specifications, the material was reviewed by both field and office engineers of the Bureau of Public Roads and by committees and individual representatives of national organizations of highway contractors and of producers and suppliers of materials and equipment. The benefits of their comments and suggestions are reflected in the book.

Standard specifications for construction of roads and bridges on Federal highway projects (FP–57) is available for purchase from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at $2.00 a copy.

First Progress Report of the Highway Cost Allocation Study

The Secretary of Commerce transmitted to the Congress on Feburary 28, 1957, a progress report on the highway cost allocation study. The report, due on or before March 1, 1957, was prepared by the Bureau of Public Roads in compliance with subsection (d) of section 210 of the Highway Revenue Act of 1956. Another progress report is due one year later, and the final report is to be made available to the Congress as soon as possible but not later than March 1, 1959.

The 131-page report, entitled *First progress report of the highway cost allocation study*, has been published as House Document No. 106, and is available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 35 cents a copy.

The objective of the study is "to provide the Congress with information on the basis of which it may determine what taxes should be imposed and in what amounts, in order to assure, insofar as practicable, an equitable distribution of the tax burden for the support of the Federal-aid highway program."

The study, which is being made in cooperation with the State highway departments and with other Federal agencies, inquires into both highway costs occasioned and the benefits derived by the use of Federal-aid highways of vehicles of different dimensions, weights, and other specifications. Inquiry will also be made into benefits, direct or indirect, that may be derived by any class of persons from public expenditures on the Federal-aid highways, other than through direct use of such highways.

The present report is in three parts: Part I is a presentation of the background of the equity problem in highway taxation; part II discusses the methods of research and analysis which will be employed in the search for equitable allocations of cost responsibility; and part III outlines the planning and organization of the study and summarizes the accomplishments to date.

Errata

The article *Applications of electrical resistivity measurements to subsurface investigations* included in the April 1957 issue of Public Roads, vol. 29, No. 7, requires a correction on page 168. The references *left* and *right* in the legend for figure 15 should be interchanged to agree with the placement of the illustrations.

U. S. GOVERNMENT PRINTING OFFICE: 1957

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Progress and Feasibility of Toll Roads and Their Relation to the Federal-Aid Program, House Document No. 139 (1955). 15 cents.
Public Control of Highway Access and Roadside Development (1947). 35 cents.
Public Land Acquisition for Highway Purposes (1943). 10 cents.
Public Utility Relocation Incident to Highway Improvement, House Document No. 127 (1955). 25 cents.
Results of Physical Tests of Road-Building Aggregate (1953). $1.00.
Roadside Improvement, No. 191MP (1934). 10 cents.
Selected Bibliography on Highway Finance (1951). 60 cents.
Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways, 1956: a reference guide outline. 55 cents.
Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, FP–57 (1957). $2.00.
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United States System of Numbered Highways. 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

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AS OF APRIL 30, 1957

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			CONTRACTS ADVERTISED, CONSTRUCTION NOT STARTED			PROJECTS UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$22,762	$35,685	$26,987	327.7	$11,224	$7,164	94.8	$74,229	$45,335	897.4	$121,138	$79,486	1,319.9
Arizona	6,230	25,652	22,390	197.2	6,943	5,764	63.0	19,535	15,886	98.5	52,130	44,040	358.7
Arkansas	37,021	19,355	11,124	474.4	11,344	5,759	105.1	35,469	21,510	425.4	66,168	38,393	1,004.9
California	9,623	57,515	33,422	235.7	50,044	39,015	45.1	391,893	181,954	318.8	499,452	254,391	599.6
Colorado	28,342	18,547	13,212	127.1	6,686	4,911	46.7	34,878	23,929	221.8	60,111	42,052	395.6
Connecticut	34,803	7,458	3,739	6.8	16,952	10,883	7.9	16,888	10,590	29.0	41,298	25,212	43.7
Delaware	17,907	3,090	1,557	26.8	1,909	1,205	12.8	14,702	8,141	63.9	19,701	10,903	103.5
Florida	24,595	26,591	20,532	217.7	18,272	13,171	40.8	46,683	26,680	316.0	91,546	60,383	574.5
Georgia	45,747	56,758	36,206	698.8	7,808	4,150	48.4	87,750	48,490	875.3	152,316	88,846	1,622.5
Idaho	30,786	7,447	5,737	89.0	1,913	1,404	18.7	15,674	10,254	199.8	25,034	17,395	307.5
Illinois	25,517	103,717	73,303	761.0	40,325	27,833	112.4	154,109	108,117	680.3	298,151	209,253	1,553.7
Indiana	86,366	27,694	15,381	111.8	13,160	7,007	95.8	41,024	25,651	142.8	81,878	48,039	350.4
Iowa	4,003	61,410	45,757	757.1	21,672	12,361	399.8	51,448	34,447	940.6	134,530	92,565	2,097.5
Kansas	11,179	53,019	42,711	990.2	8,242	4,503	183.7	40,911	23,955	1,135.1	102,172	71,169	2,309.0
Kentucky	57,037	8,371	5,909	62.3	1,250	814	5.5	43,044	26,959	330.5	52,665	33,682	398.3
Louisiana	39,436	34,779	17,896	192.1	15,149	8,220	60.3	42,494	22,808	274.8	92,422	48,924	527.2
Maine	24,955	11,621	6,209	91.7	1,150	680	8.4	19,128	9,972	119.2	31,899	16,861	219.3
Maryland	6,773	23,020	11,672	131.8	13,575	10,614	10.2	60,068	39,995	160.3	96,663	62,281	302.3
Massachusetts	37,580	35,282	21,522	32.5	43,246	25,733	29.8	55,896	30,966	48.1	134,424	78,221	110.4
Michigan	17,027	83,064	62,818	648.0	44,966	27,020	98.9	94,753	63,397	338.5	222,783	153,235	1,085.4
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Mississippi	11,356	41,915	29,866	708.8	20,702	17,158	128.6	33,306	19,083	670.9	95,923	66,107	1,508.3
Missouri	29,343	43,532	27,533	1,269.3	11,262	9,026	25.4	111,299	69,708	1,092.9	166,093	106,267	2,387.6
Montana	38,567	9,338	5,746	185.6	7,934	4,823	117.4	46,344	33,132	437.4	63,616	43,701	740.4
Nebraska	38,636	27,994	16,677	444.8	5,577	3,145	108.6	33,853	19,279	984.0	67,424	39,101	1,537.4
Nevada	29,385	6,655	5,958	70.1	2,331	1,919	15.9	15,019	12,809	196.4	24,005	20,686	282.4
New Hampshire	11,261	12,126	8,608	31.5	1,063	530	6.4	17,073	10,227	71.7	30,262	19,365	109.6
New Jersey	72,862	18,622	10,431	75.9	8,821	4,686	14.5	42,864	24,638	33.5	70,307	39,755	123.9
New Mexico	10,891	9,928	7,919	72.6	7,683	6,240	46.0	32,511	25,773	246.1	50,122	39,932	364.7
New York	87,828	52,616	31,306	157.2	90,778	59,349	83.1	354,488	203,567	425.5	497,882	294,222	665.8
North Carolina	60,549	26,390	16,407	320.4	13,226	10,171	83.3	67,584	34,391	835.6	107,200	60,969	1,239.3
North Dakota	13,386	24,237	17,347	1,407.1	13,303	9,145	312.0	22,555	13,399	957.4	60,095	39,891	2,676.5
Ohio	38,016	92,742	65,374	191.9	14,654	10,569	12.6	165,287	108,768	228.5	272,683	184,711	433.0
Oklahoma	27,470	46,285	32,457	571.1	10,057	5,584	88.3	44,520	26,086	398.7	100,862	64,127	1,058.1
Oregon	23,627	7,404	5,976	66.5	2,844	2,460	16.0	41,611	29,666	240.4	51,859	38,102	322.9
Pennsylvania	36,605	117,886	81,657	205.3	83,704	55,975	78.5	163,870	97,930	333.9	365,460	235,562	617.7
Rhode Island	6,223	2,032	1,128	2.6	10,079	8,670	10.3	26,187	16,707	19.8	38,298	26,505	32.7
South Carolina	36,418	22,753	14,727	477.2	2,051	1,483	10.7	36,399	20,566	703.6	61,203	36,776	1,191.5
South Dakota	11,597	32,607	24,539	607.7	5,236	2,929	141.8	26,477	17,429	769.0	64,320	44,897	1,518.5
Tennessee	45,715	30,790	19,144	474.4	12,242	6,250	33.1	81,250	45,654	582.6	124,282	71,048	1,090.1
Texas	93,180	30,323	24,023	271.2	60,370	42,656	239.0	160,820	99,025	1,720.1	251,513	165,704	2,230.3
Utah	16,380	17,114	14,265	156.9	1,654	1,335	2.1	14,938	11,913	179.7	33,706	27,513	338.7
Vermont	11,442	3,753	2,676	32.1	3,740	2,477	5.9	16,844	11,298	63.4	24,337	16,451	101.4
Virginia	41,533	41,428	28,898	177.5	7,915	5,318	21.9	44,993	24,272	371.3	94,336	58,488	570.7
Washington	36,130	15,280	9,883	277.8	8,210	6,545	51.4	43,089	26,650	270.5	66,579	43,078	599.7
West Virginia	23,336	38,873	27,200	64.5	4,741	2,393	25.9	30,078	16,234	82.2	73,692	45,827	172.6
Wisconsin	53,967	21,216	11,682	401.5	6,362	3,273	28.3	65,914	38,833	410.5	93,492	53,788	840.3
Wyoming	1,046	15,709	13,147	81.3	15,180	12,647	112.0	27,251	20,066	331.0	58,140	45,860	524.3
Hawaii	5,969	3,246	1,623	11.0	3,157	1,575	4.5	4,935	2,373	9.0	11,338	5,571	24.5
District of Columbia	15,394	13,242	10,504	8.4	3,172	1,904	2.0	11,210	8,487	.1	27,624	20,895	10.5
Puerto Rico	10,789	5,638	2,303	15.8				18,949	9,044	61.1	24,587	11,347	76.9
Alaska	1,226	14,589	13,130	413.8	80	72	9.0	653	606		15,322	13,808	422.8
TOTAL	1,535,292	1,574,330	1,080,352	16,372.7	787,084	526,639	3,560.4	3,209,375	1,922,290	21,072.1	5,570,789	3,529,281	41,005.2

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In this issue: A study of vehicle use of traffic lanes and shoulders on two-lane primary rural highways.

Public Road

A JOURNAL OF HIGHWAY RESEARC

Published Bimonthly

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U. S. DEPARTMENT OF COMMERCE
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Driver Behavior Related to Types and Widths of Shoulders on Two-Lane Highways

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH BUREAU OF PUBLIC ROADS

Reported [1] by ASRIEL TARAGIN, Head,
Driver Behavior Section

In this article the results of a study of vehicle placements and speeds observed on two-lane primary rural highways are discussed. Speed and placement data were recorded for 89,000 vehicles at 66 locations in nine Western States.

This analysis is concerned primarily with bituminous pavements, although a limited number of sections paved with portland cement concrete were included in the study.

Lateral position and speed studies were conducted on test sections with various types and widths of shoulders—gravel shoulders, combinations of bituminous materials and gravel, and bituminous-paved shoulders. On some sections of bituminous pavements and shoulders, there was sufficient contrast in the appearance of the traffic lanes and shoulders for drivers to differentiate between them; on other sections, the bituminous-paved traffic lanes and shoulders appeared to be a continuously paved roadway.

Commercial vehicles made much greater use of the shoulders than passenger cars. On test sections with matching bituminous-paved traffic lanes and shoulders, nearly 80 percent of the commercial vehicles meeting other vehicles traveled partly on the shoulder. The use of a 2-inch solid white stripe to separate the traffic lane from the shoulder had the effect of reducing shoulder encroachment by about 50 percent.

Shoulder surface types of comparable widths did not seem to influence vehicle speeds. Passenger-car speeds for all locations studied averaged 55 miles per hour and commercial vehicles, 48 miles per hour.

APPROXIMATELY 94 percent of the existing mileage of primary rural roads in the United States were designed to accommodate two lanes of traffic. In the Western States, the greatest proportion of these roads are surfaced with bituminous materials. The normal practice in the past was to construct gravel shoulders adjacent to the traveled anes, but in recent years, a number of the Western States have adopted the practice of also paving the shoulders with bituminous materials.

In most cases there has been a definite distinction between the appearance of the traveled lanes and the shoulders, either in color or texture or both. On a small but significant mileage of these rural roads, however, it is impossible to distinguish between traffic lanes and shoulders. These roads appear to be two-lane highways with 20-foot anes and no shoulders instead of the normal 12-foot lanes with 8-foot shoulders. Constructing highways in this manner has occasioned much concern among engineers not only because shoulder areas are not usually designed to carry the loads of the traffic lanes, but drivers tend to operate on these sections as if they were four-lane undivided highways.

This concern resulted in a study of driver behavior and the effect of contrasting traffic lanes and shoulders on safety of operation as compared with construction practices where shoulders cannot be distinguished from traffic lanes. The study was a series of cooperative undertakings between the Bureau of Public Roads and the State highway departments of Arizona, California, Colorado, Idaho, New Mexico, Oregon, Texas, Utah, and Washington. Field work was started in March 1955 and continued for 6 months. Vehicle speeds and placements were recorded for each of the several sections selected for study, and passing maneuvers were observed over a one-half mile length of highway at a limited number of locations. Observers noted the types of vehicles involved in each passing maneuver and their approximate transverse positions.

Summary of Major Findings

The results of this study point to a number of significant aspects of driver behavior with respect to the use of shoulders on rural two-lane highways carrying light to moderate traffic volumes. Included in the study were road sections with gravel shoulders adjacent to paved traffic lanes, bituminous-paved shoulders contrasting in appearance with the paved traffic lanes, and bituminous-paved shoulders matching the paved traffic lanes. Several types of edge striping were investigated to determine their effect on the lateral positions of vehicles traveling on the test sections.

The more important findings of this study follow:

1. Although vehicle speeds were not affected by shoulder surface types, a relation did exist between speeds and lateral positions of vehicles on test sections with matching bituminous-paved traffic lanes and shoulders. The average position of the slower moving vehicles, whether passenger cars or trucks, was closer to the shoulder of the highway than that of the faster moving vehicles.
2. Commercial vehicles encroached on shoulders to a greater extent than passenger cars. This was particularly apparent on bituminous-paved sections with matching traffic lanes and shoulders where nearly 80 percent of the trucks meeting other vehicles traveled partly on the shoulder. On sections with contrasting traffic lanes and shoulders, encroachment on shoulders was one-third to one-half that of sections with matching traffic lanes and shoulders.
3. Average clearances between bodies of meeting vehicles were about 6 feet on test sections with wide gravel shoulders, about 7½ feet on sections with a combination of bituminous and gravel shoulders, and over 10 feet on bituminous-paved sections with matching traffic lanes and shoulders.
4. Nearly 30 percent more passing maneuvers were observed on test sections with matching traffic lanes and shoulders in comparison with all other combinations of contrasting lanes and shoulders.
5. In the study of lateral positions of vehicles, it was evident that the greater the degree of contrast in appearance between traffic lanes and shoulders the greater the tendency for drivers to operate within the confines of the traffic lanes. This concentra-

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[1] This article was presented at the 36th Annual Meeting of the Highway Research Board, Washington, D. C., January 1957.

Table 1.—Number of two-lane highway test sections included in study, classified according to surface types and widths of traffic lanes and shoulders

Group	Number of locations	Number of vehicles	Traffic lanes		Shoulder width		Appearance of bituminous shoulder compared with traffic lanes
			Width	Type	Bituminous	Gravel	
			Feet		*Feet*	*Feet*	
1	3	4, 000	12	Bituminous		3–5	
2	8	8, 900	12	do		6–10	
3	6	9, 900	12	do	4	4–6	Contrasting.
4	15	22, 800	12	do	6–10		Do.
5	16	14, 600	12	do	8		No eontrast.
6	3	4, 900	12	Portland cement concrete	8–10		
7	3	5, 100	12	do		9	
8	4	7, 900	11	Bituminous	6–8		Contrasting.
9	3	2, 400	12	do	8		No contrast.[1]
10	2	2, 600	12	do	8		No contrast.[2]
11	3	5, 900	12	do	4	6	Contrasting.[2]

[1] Stripe near outside edge of shoulder.
[2] Stripe near outside edge of traffic lane.

Table 2.—Average speeds of vehicles traveling on two-lane bituminous-surfaced rural highways with 12-foot traffic lanes and shoulders of various surface types and widths

Vehicle classification	Average speeds for two-lane highways with—				
	Gravel shoulders, 3–5 feet	Gravel shoulders, 6–10 feet	Bituminous and gravel shoulders, 8–10 feet [1]	Bituminous shoulders, 6–10 feet [2]	Bituminous shoulders, 8 feet [3]
Passenger cars:	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*
Free moving	53. 4	58. 2	55. 9	56. 3	54. 3
Meeting other vehicles	53. 2	57. 2	55. 0	56. 0	53. 3
All other passenger cars	52. 5	56. 6	54. 0	55. 4	53. 2
Commercial vehicles:					
Free moving	46. 2	49. 3	47. 2	48. 4	47. 5
Meeting other vehicles	45. 5	50. 1	48. 7	48. 2	49. 1
All other eommercial vehieles	46. 7	48. 6	46. 6	48. 1	46. 7

[1] Four feet of bituminous eontrasting with traffic lane and 4 to 6 feet of gravel.
[2] Contrasting with traffic lane.
[3] No eoutrast with traffic lane

tion of lateral placement was particularly noticeable on bituminous and concrete test sections with gravel or grass shoulders.

6. Shoulder edge stripes closer than 1½ feet from the outside edge of bituminous-paved shoulders (18½ feet from the centerline of the roadway) had no effect on vehicle speeds or lateral positions.

7. A 2-inch solid white stripe painted 8 feet from the outside edge of the shoulder or 12 feet from the centerline of the roadway having matching bituminous-paved lanes and shoulders had the effect of reducing shoulder encroachment by about 50 percent as compared to test sections without edge stripes.

8. On bituminous pavements with combination-type shoulders of 4 feet of bituminous material and 6 feet of gravel, a 4-inch solid yellow stripe placed 13 feet from the centerline of the roadway was very effective in reducing shoulder use, especially by trucks.

9. On two-lane, 22-foot bituminous pavements with contrasting 6- to 8-foot bituminous shoulders the lateral positions of vehicles and clearances between meeting vehicles were about the same as for two-lane, 24-foot bituminous pavements with 6- to 10-foot gravel shoulders.

Locations Studied

Only level tangent sections having a cross-section design typical of that used in the particular State were included in the program. The number of locations studied in each of the several groups of roadway sections is shown in table 1. Speed and placement data were obtained for 89,000 vehicles at 66 locations during the daylight hours; data were also recorded at night at the locations included in groups 9–11.

The average speed and placement data for the several groups of locations are representative of driver behavior for the cross-section designs shown. Even though the speeds and placements at some locations with a given cross section were somewhat different fr those at other locations having the same cr section, the variation was remarkably sm

The information recorded for groups 1–5 of primary interest insofar as bituminous su- faces are concerned. These five groups rep- sent five different shoulder widths and surfa- type combinations on two-lane highwa having 12-foot bituminous-surfaced lanes. more detailed description of the types shoulders included in these groups follows:

Group 1.—Gravel shoulders, 3 to 5 fe wide. The material consisted of gravel loose stone chips having the same appeara as gravel. The cross slope of the shoulder w considerably greater than the traffic lanes.

Group 2.—Gravel shoulders, 6 to 10 fe wide. The cross sections, other than the wid of the shoulder, were approximately the sa as group 1. Included also were should treated with oil and covered with crush gravel.

Group 3.—Four-foot bituminous-pav shoulders contrasting in appearance with t traffic lanes. Gravel shoulders, 4 to 6 fe wide, extended beyond the paved shoulder.

Group 4.—Bituminous-paved shoulders, to 10 feet wide, contrasting with the traf lanes. Beyond the paved shoulder, the roa- bed sloped sharply to the ditch line. On se eral sections the slope of the shoulder w noticeably greater than that of the traffic la

Group 5.—Bituminous-paved shoulders, feet wide, having the same appearance, t ture, and riding qualities as the traffic la To all outward appearance these roads we two-lane highways with 20-foot lanes and shoulders.

Groups 6–11.—These groups include cr sections of varied shoulder widths and su face types, some of which are typical in certa States. Special studies of edge striping a signs were conducted on sections included these groups.

Table 3.—Percentages of vehieles traveling below 40 and above 60 miles per hour on tw lane bituminous-surfaced rural highways with 12-foot traffie lanes and shoulders various surface types and widths

Vehicle classification	Percentage of vehicles traveling at indicated speeds on two-lane highways with—				
	Gravel shoulders, 3–5 feet	Gravel shoulders, 6–10 feet	Bituminous and gravel shoulders, 8–10 feet [1]	Bituminous shoulders, 6–10 feet [2]	Bituminou shoulders, 8 feet [3]
VEHICLES TRAVELING BELOW 40 MILES PER HOUR					
Passenger cars:	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Free moving	10. 4	6. 4	6. 6	6. 7	8. 6
Meeting other vehicles	11. 6	5. 8	7. 7	7. 2	11. 6
All other passenger cars	14. 2	7. 2	8. 9	7. 5	9. 6
Commercial vehicles:					
Free moving	25. 9	14. 9	19. 3	14. 8	20. 8
Meeting other vehicles	29. 1	10. 5	19. 1	18. 4	16. 9
All other commercial vehicles	25. 0	13. 2	18. 6	17. 0	24. 4
VEHICLES TRAVELING ABOVE 60 MILES PER HOUR					
Passenger cars:					
Free moving	29. 6	45. 7	39. 3	42. 0	31. 6
Meeting other vehicles	27. 9	41. 3	34. 9	40. 9	26. 3
All other passenger cars	29. 0	39. 0	31. 2	38. 6	28. 9
Commercial vehicles:					
Free moving	9. 0	12. 3	9. 3	10. 5	11. 0
Meeting other vehicles	3. 5	15. 4	13. 5	10. 9	12. 8
All other eommercial vehicles	9. 7	11. 1	7. 3	10. 2	9. 9

[1] Four feet of bituminous contrasting with traffic lane and 4 to 6 feet of gravel.
[2] Contrasting with traffic lane.
[3] No contrast with traffic lane.

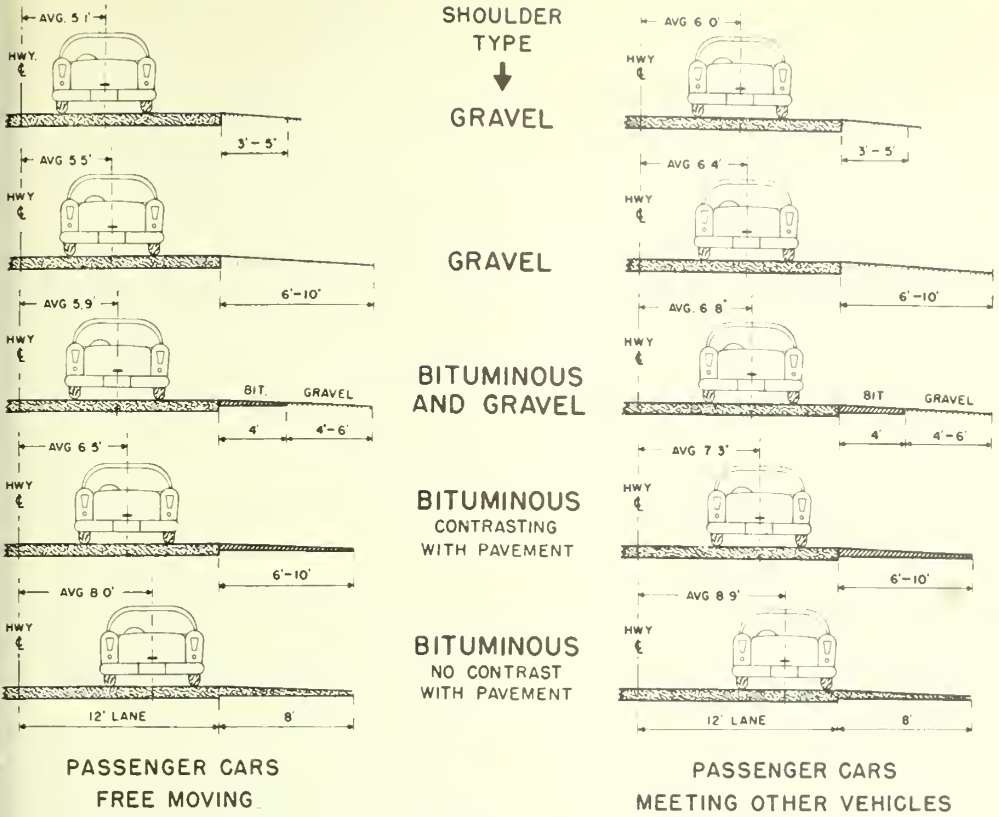


Figure 1.—Average lateral positions of passenger cars on two-lane bituminous-paved rural highways.

Classification of Data

The data collected consisted primarily of speeds and placements of all vehicles as they passed a selected point of observation. Each vehicle was classified as to whether it was free moving, meeting another vehicle traveling in the opposite direction, trailing a vehicle in the same direction, passing another vehicle, or being passed. A vehicle not belonging in any of the foregoing groups was classified as "other." It has been found that the best measure of the effect on driver behavior of the variables under study is provided by the free-moving and meeting vehicles. Reliable comparisons of speeds and placements were possible between vehicles in these two groups. The remaining groups, with the exception of vehicles passing other vehicles traveling in the same direction, were combined into one group and identified as "all other vehicles." The number of vehicles involved in passing maneuvers during the periods of study were too few to permit an analysis which would produce significant results. Because of their position in the left traffic lane (negative placement), the passing vehicles were not included in any of the groups.

Comparative data are therefore shown for vehicles in the following three groups:

Free-moving vehicles.—Those vehicles which were, for practical purposes, uninfluenced by other traffic on the highway when speeds and transverse positions were recorded. About 55 percent of the vehicles studied were in this group.

Meeting vehicles.—Those vehicles that might have been directly affected by opposing traffic but not by vehicles traveling in the same direction. Clearances between the bodies of these vehicles were calculated from the placement data. About 15 percent of the vehicles were in this group.

All other vehicles.—Those vehicles which were not classified as free moving or meeting other vehicles. These vehicles, which constituted 30 percent of all traffic, probably included some that were influenced in speeds and placements by other traffic.

Speeds and placements were analyzed separately for passenger cars and commercial vehicles at each location studied. Data obtained at the several sites having identical highway cross-section geometry were then combined.

Vehicle Speeds

Table 2 shows the average speeds of vehicles recorded at 48 locations included in groups 1–5. There is no great variation between the speeds on sections with shoulders of approximately the same width. A comparison of speeds observed on sections with gravel shoulders, averaging 4 and 8 feet wide, indicates that the various classes of vehicles traveled about 2 to 5 miles per hour faster on sections with the wider shoulders. Passenger-car speeds on all sections averaged 55 miles per hour and trucks averaged 48 miles per hour.

Table 3 provides a comparison of the percentages of vehicles traveling below 40 miles per hour and exceeding 60 miles per hour. Again, it is evident that shoulder surface types of approximately the same width did not influence speeds significantly.

Vehicle Placements

The average lateral positions of free-moving passenger cars and of passenger cars meeting other vehicles are shown in figure 1. Free-moving passenger cars maintained an average lateral position progressively farther from the centerline of the highway as the shoulder was widened and improved in type. On sections with 6- to 10-foot gravel shoulders, the average

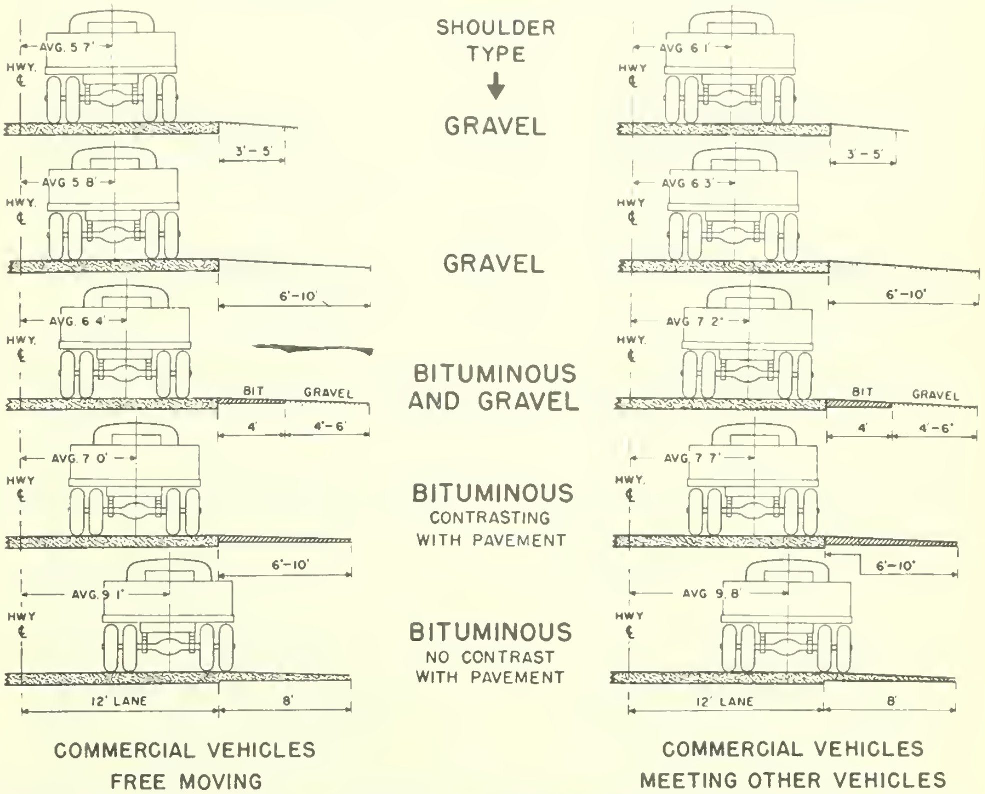
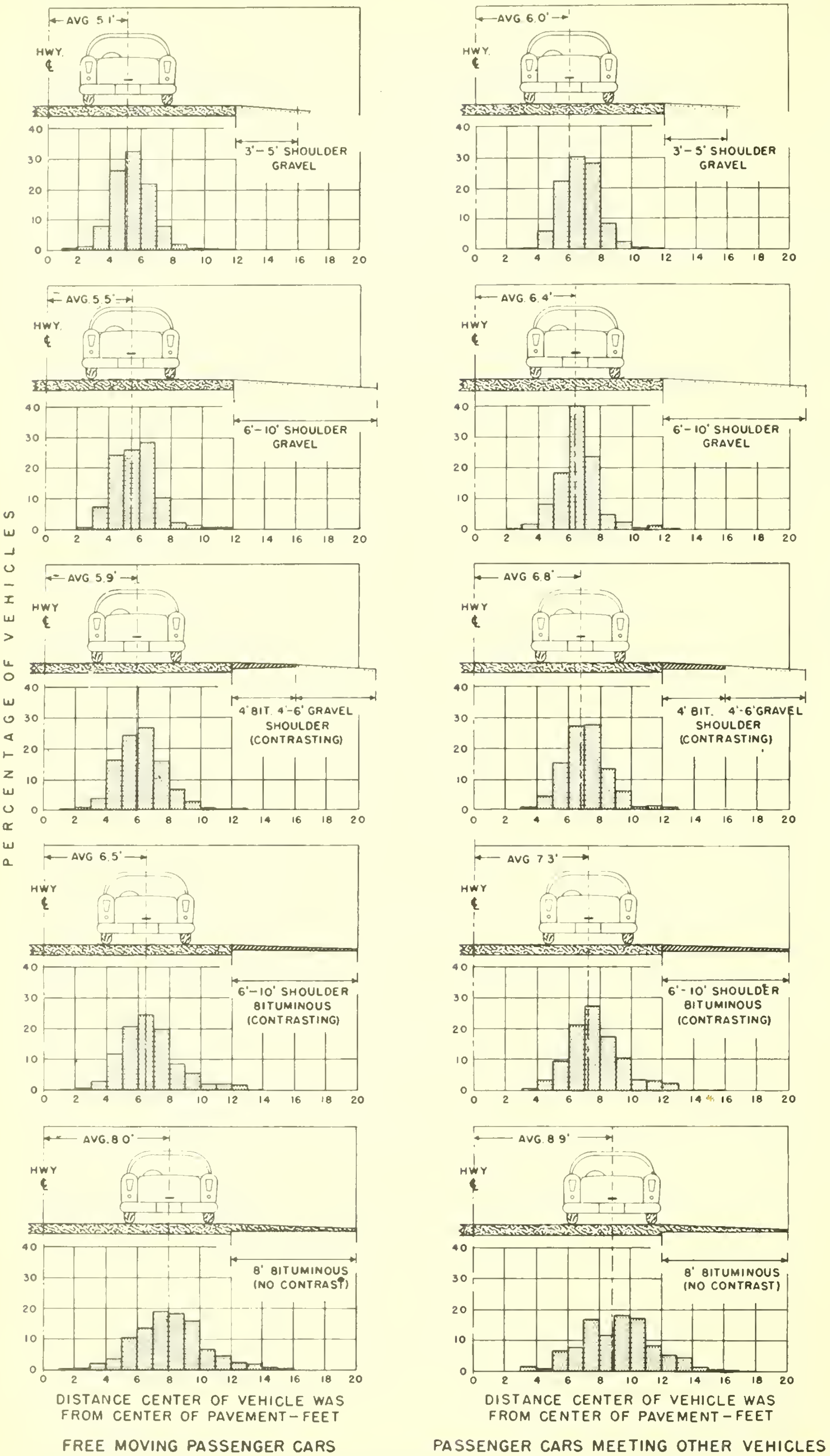


Figure 2.—Average lateral positions of commercial vehicles on two-lane bituminous-paved rural highways.

position was 0.4 foot farther to the right than on sections with 3- to 5-foot gravel shoulders. On sections with 4-foot contrasting bituminous shoulders, and with an additional 4 to 6 feet of gravel outside the paved portion, the average position was 0.4 foot farther to the right than on the sections with 6- to 10-foot grave shoulders. In this progressive trend, th greatest difference in the average lateral position occurred between sections with matching shoulders and traffic lanes as compared with sections with contrasting shoulders and traffic lanes. The difference between the lateral positions of free-moving passenger cars for these two groups was 1.5 feet.



FREE MOVING PASSENGER CARS

PASSENGER CARS MEETING OTHER VEHICLES

Figure 3.—Distribution of lateral positions of passenger cars on two-lane bituminous-paved rural highways.

Some highway engineers have expressed the view that an ideal cross section might be one where the average free-moving vehicle travels in the same path as vehicles meeting oncoming traffic. In other words, traffic would assume a certain lateral position and maintain that position even when meeting oncoming vehicles. It is noted in figure 1, however, that free-moving passenger cars travel 0.8 or 0.9 foot nearer the centerline than passenger cars meeting other vehicles for all shoulder widths and types studied.

As indicated in figure 2, free-moving commercial vehicles on the average traveled 0.4 to 0.5 foot closer to the centerline than those meeting other vehicles on sections with grave shoulders; on the other sections, the difference was 0.7 to 0.8 foot.

Figure 3 shows the distribution of latera positions of free-moving passenger cars and passenger cars meeting other vehicles. This illustration is similar to figure 1, except that it shows the distribution of the lateral positions. The positions were determined by measuring the distance from the center of the car to the centerline of the pavement A value less than 3 feet from the centerline of the highway indicates that the car was encroaching on the left traffic lane. Similarly a value greater than 9 feet indicates that the car was encroaching on the shoulder area Figure 3 shows that on two-lane, 24-foot bituminous pavements with gravel shoulders the lateral positions are concentrated mainly within a 3-foot strip in the center of the traffic lane. On sections with 8-foot matching shoulders, the lateral positions are distributed over a distance of about 6 feet.

Encroachment on left lane and on shoulder

The percentages of vehicles encroaching on the left lane and on the shoulder are shown in table 4. It is evident that the type of shoulder does not materially affect the percentages of vehicles straddling the centerline. In general few vehicles straddle the centerline, and this is particularly true of vehicles meeting other traffic. This indicates that 12-foot lanes are adequate for two-lane rural highways and supports the findings of a previous study.[2]

Shoulder use, as shown in table 4, is definitely related to the shoulder type. As might be expected, vehicles meeting other traffic encroach on the shoulder to a larger extent than vehicles in any of the other groups Commercial vehicles make greater use of the shoulders than passenger cars. The degree of encroachment on the shoulder under various conditions is illustrated in figure 4.

Encroachment on the shoulder area was greatest on sections with matching pavement

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[2] *Effect of roadway width on vehicle operation*, by A. Taragin PUBLIC ROADS, vol. 24, No. 6, Oct.–Nov.–Dec. 1945.

Table 4.—Percentages of vehicles straddling centerline or traveling on shoulders of two-lane bituminous-surfaced rural highways with 12-foot traffic lanes and shoulders of various surface types and widths

Vehicle classification	Percentage of vehicles traveling at indicated lateral positions on two-lane highways with—				
	Gravel shoulders, 3–5 feet	Gravel shoulders, 6–10 feet	Bituminous and gravel shoulders, 8–10 feet [1]	Bituminous shoulders, 6–10 feet [2]	Bituminous shoulders 8 feet [3]
VEHICLES STRADDLING CENTERLINE					
Passenger cars:	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Free moving	1.3	0.6	1.0	0.4	0.4
Meeting other vehicles	.2	.1	.3	.2	0
All other passenger cars	2.8	1.4	1.2	.9	.9
Commercial vehicles:					
Free moving	2.5	3.9	4.9	.9	1.3
Meeting other vehicles	0	3.1	1.0	1.5	.5
All other commercial vehicles	3.1	2.0	5.1	1.8	1.4
VEHICLES TRAVELING ON SHOULDER					
Passenger cars:					
Free moving	0.7	1.7	3.8	11.6	33.8
Meeting other vehicles	3.4	3.3	10.0	19.8	55.1
All other passenger cars	.6	1.9	5.8	12.1	38.1
Commercial vehicles:					
Free moving	6.1	3.9	16.4	27.8	67.3
Meeting other vehicles	8.3	10.5	31.6	40.7	78.4
All other commercial vehicles	4.6	5.7	24.5	30.8	68.8

[1] Four feet of bituminous contrasting with traffic lane and 4 to 6 feet of gravel.
[2] Contrasting with traffic lane.
[3] No contrast with traffic lane.

nd shoulders. On these sections nearly 80 ercent of the trucks meeting other vehicles aveled partly on the shoulder. On the asis of these findings, it would appear that here is justification for placing a uniform ibgrade and asphalt pavement over the ntire width of the roadway. Arizona, for xample, has followed this practice for a umber of years. The data in figure 4 indicate, however, that if there is a contrast etween traffic lanes and shoulders, the ructural strength of the entire width of the ioulder need not be as great as that of the aveled lane. Shoulder encroachment on ctions with contrasting shoulders is one-third one-half that of sections with matching ivement and shoulders.

learances Between Meeting Vehicles

Figure 5 shows the clearances between odies of passenger cars meeting other issenger cars for the various types and widths shoulders. The percentage of vehicles iving clearances of less than 4 feet was ry small. On sections with narrow gravel oulders, 3 to 5 feet, the average clearance tween bodies of passenger cars meeting her passenger cars was 5.8 feet. The erage clearances increased with the wider d higher type shoulders. On sections with atching pavement and shoulders, the clearces between bodies of meeting passenger rs averaged 11.1 feet, with 94 percent of e vehicles having clearances of 6.0 feet or ore. It is noteworthy that clearances of 10 t or more were recorded in 63 out of 100 servations on matching pavement and oulder sections, whereas on sections with ivel shoulders, clearances of 10 feet or more re recorded in 3 out of 100 observations. A cumulative distribution of clearances tween bodies of passenger cars meeting mmercial vehicles is shown in figure 6. Clearances were least on sections with gravel shoulders. On sections with 3- to 5-foot gravel shoulders, 16 percent of the vehicles had clearances of 4 feet or less. On sections with 4-foot bituminous-paved shoulders and 4- to 6-foot gravel surfaces outside the paved portion, only 3 percent of the clearances were below 4 feet. On sections where the shoulders were paved with 6 to 10 feet of contrasting bituminous material, the clearances were only slightly greater than on sections with 4 feet of bituminous material in combination with 4 to 6 feet of gravel. It is obvious that on sections with matching bituminous shoulders, the clearances between passenger cars meeting commercial vehicles are greater than needed for safe operation.

Average clearances between the bodies of commercial vehicles meeting other commercial vehicles were obtained at several locations and the results are shown in the last column of table 5. Although the sample was small, it is interesting to note that only on sections with contrasting pavement and shoulders were the clearances between the bodies of meeting commercial vehicles about the same as for passenger cars.

Sections with oil-penetration shoulders covered with crushed gravel were included with gravel shoulder sections of the same width. Driver behavior followed the same pattern for both treated and untreated gravel shoulders. This was to be expected because drivers could not distinguish between the two types.

Relation Between Speed and Placement

On sections of highways having the usual 12-foot traffic lanes with grass or gravel shoulders, studies heretofore have shown that there is very little relation between the speeds of vehicles and their lateral positions in the traffic lane. Because of the many sections with paved shoulders included in this study, an effort was made to determine whether any relation exists between the speed of a vehicle and its lateral position on the highway.

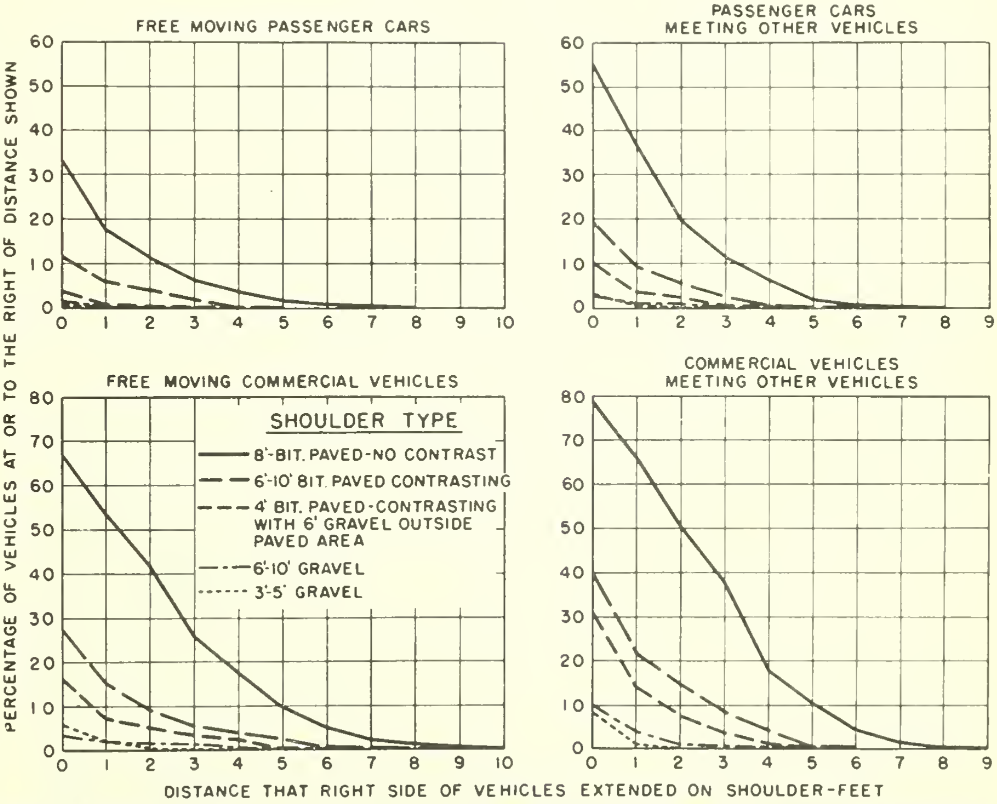

Figure 4.—Shoulder encroachment of passenger cars and commercial vehicles on two-lane bituminous-paved rural highways.

A typical illustration of the results is shown in figure 7. This figure shows the average position of free-moving passenger cars traveling at various speeds on two-lane rural roads with various shoulder widths and types. On sections with gravel shoulders, there was only a very slight tendency for the slower moving vehicles to travel closer to the shoulder area than the faster moving vehicles. The tendency was much greater, however, on paved shoulder sections. On sections where the pavement and shoulders were uniform in appearance, the lateral position of the slowest group of drivers was more than 2.0 feet closer to the shoulder than the position of the fastest group. In other words, the slower moving passenger cars utilized the full width of paving to a greater degree than the faster vehicles. Relations similar to those shown in figure 7 for free-moving passenger cars were found to exist for other groups of passenger cars and commercial vehicles.

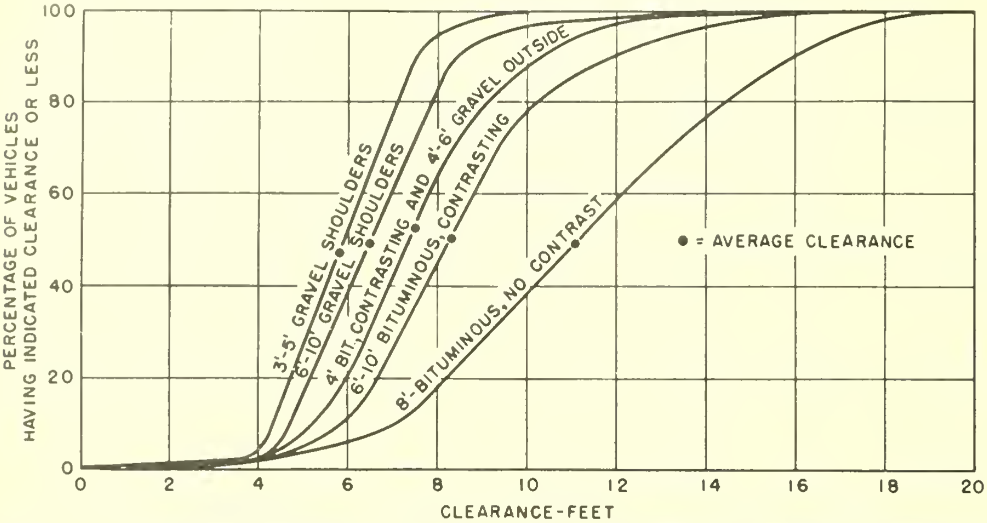

Figure 5.—Cumulative distribution of clearances between bodies of meeting passenger cars, as related to shoulder widths and surface types on two-lane bituminous-paved rural highways.

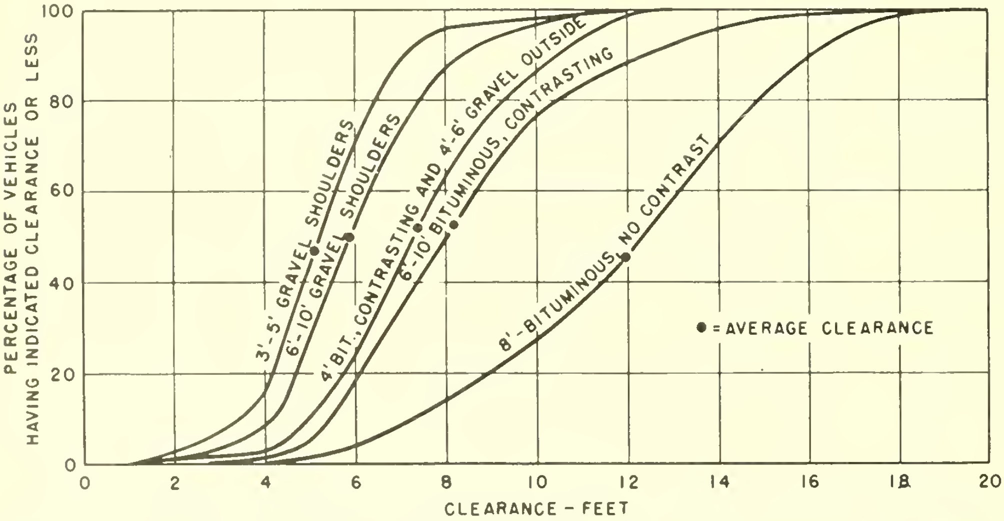

Figure 6.—Cumulative distribution of clearances between bodies of passenger cars meeting commercial vehicles, as related to shoulder widths and surface types on two-lane bituminous-paved rural highways.

Table 5.—Average clearances between bodies of meeting vehicles traveling on two-lane bituminous-surfaced highways with 12-foot traffic lanes and shoulders of various surface types and widths

Shoulder type and width	Average clearance between bodies of—		
	Passenger cars meeting other passenger cars	Passenger cars meeting commercial vehicles	Commercial vehicles meeting other commercial vehicles
	Feet	*Feet*	*Feet*
Gravel shoulders, 3–5 feet	5.8	5.1	4.6
Gravel shoulders, 6–10 feet	6.5	5.9	5.2
Bitiminous and gravel shoulders, 8–10 feet [1]	7.5	7.4	7.6
Bituminous shoulders, 6–10 feet [2]	8.3	8.2	8.1
Bituminous shoulders, 8 feet [3]	11.1	11.9	10.4

[1] Four feet of bituminous contrasting with traffic lane and 4 to 6 feet of gravel.
[2] Contrasting with traffic lane.
[3] No contrast with traffic lane.

Passing Practices

In addition to speed and placement studies, passing maneuvers were observed over a one-half mile length of highway at 27 locations. The observers classified each passing maneuver according to the relative transverse and longitudinal positions of the passed and passing vehicles, as shown in table 6. The number of passings per mile per hour on sections with combination-type shoulders (bituminous material and gravel) was the same as on sections with contrasting bituminous shoulders and traffic lanes. On bituminous-paved sections with matching shoulders and traffic lanes, the number of passings performed, reduced to a common traffic volume, was about 30 percent higher than on the other two cross sections.

It appears therefore that matching traffic lanes and shoulders tend to encourage passing maneuvers. Three-lane operation was almost nonexistent on sections having shoulders surfaced with a combination of gravel and bituminous material.

Concrete Pavement Operation

As previously mentioned, shoulder usage was less on bituminous pavements with contrasting shoulders and traffic lanes than on sections with matching shoulders and traffic lanes. The shoulder usage was further reduced on concrete pavements with bituminous shoulders, and still further reduced on sections with grass or gravel shoulders.

Speed and placement data were recorded on six level tangent sections of two-lane, 24-foot concrete pavements. On three of these sections the pavement was flanked by 9-foot bituminous-paved shoulders, and on the other three sections the shoulders were grass or gravel measuring 8 to 10 feet wide. Table 7 compares the results obtained on sections having bituminous shoulders with those obtained on sections with grass or gravel shoulders. The latter group of sections is identified in table 7 as gravel shoulders, since results of an earlier study [3] as well as the results of the present study indicate that well-maintained grass shoulders have the same effect on the lateral position of moving vehicles as well-maintained gravel shoulders.

The results shown in table 7 indicate that the speeds were about the same on the road sections with gravel shoulders as on the sections with bituminous shoulders. Passenger cars traveled about one foot closer to the bituminous shoulders than to the gravel shoulders, while commercial vehicles maintained about the same average lateral position for both shoulder types.

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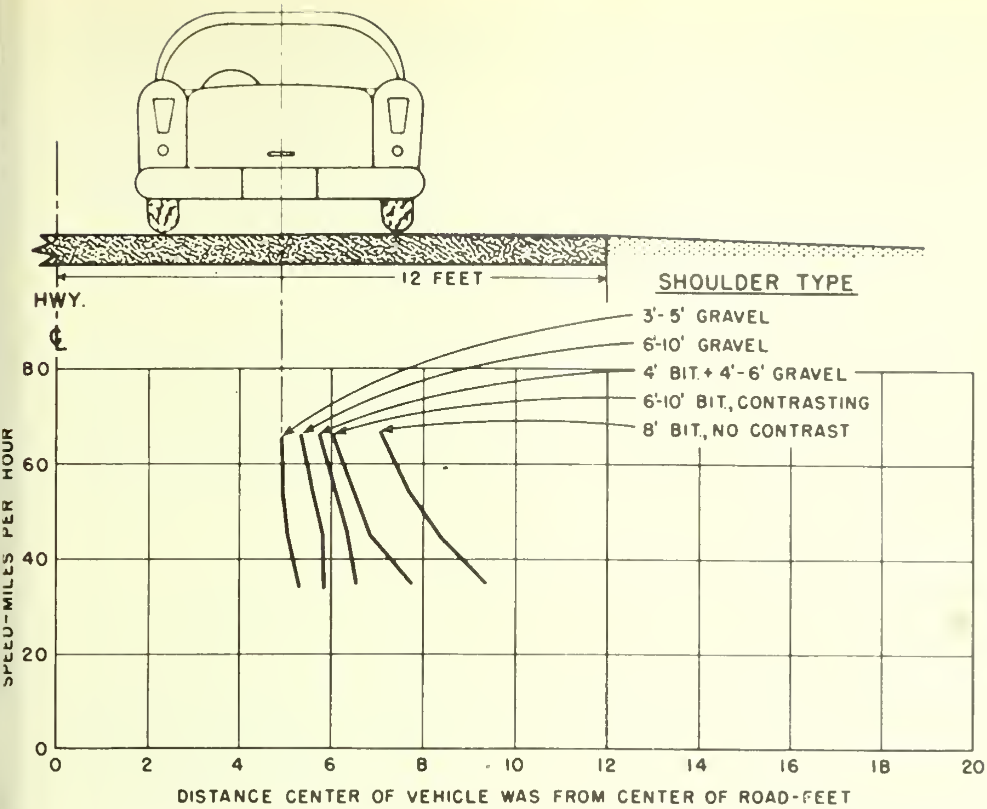
[3] See footnote 2, p. 200.



'igure 7.—*Average lateral positions of free-moving passenger cars traveling at various speeds on two-lane bituminous-paved rural highways.*

Although the percentages of vehicles strad-ling the centerline were small for either type f shoulder, passenger cars straddled the enterline on sections with gravel shoulders ıore often than on sections with bituminous houlders. On the other hand, results show n opposite trend for commercial vehicles, lthough the difference was slight.

A comparison of the results shown in table 7 or concrete pavements and similar results for ituminous pavements, shown in figures 1–2, eveals that bituminous shoulders adjacent to ıe concrete traffic lanes were more effective ı confining vehicles to the traffic lane than ituminous pavements with contrasting bitu-ninous shoulders. Vehicle operation, as ıeasured by lateral positions and clearances etween vehicles, on two-lane, 24-foot concrete ıvements with 9-foot bituminous shoulders as about the same as that for two-lane, 4-foot bituminous pavements with a com-nation of 8- to 10-foot bituminous and avel shoulders. It is evident that the eater the contrast in appearance between ıe traffic lane and the paved shoulder, the etter the positioning of the moving vehicle ithin the traffic lane.

Substandard Pavement Widths

Although 11-foot traffic lanes are not now e standard width for primary two-lane high-ıys, there is a considerable mileage of such ads. In conjunction with the studies on ctions with 12-foot lanes, four locations with -foot lanes were studied in three States. ıese pavements were flanked by 6- to 8-foot tuminous shoulders. One of the locations Oregon had red paved shoulders adjacent the traffic lanes. Another location, also in Oregon, had black shoulders adjacent to a red pavement. Red aggregate with an asphaltic binder was used to get this color contrast. On the other two locations in California and Washington, the shoulders and traffic lanes were bituminous surfaced with a definite contrast in appearance.

Vehicle speeds on these 22-foot pavements were about the same as for the other sections of two-lane roads with wider surfaces. Lateral positions with respect to the highway centerline and clearances between meeting vehicles on these 22-foot sections also followed a similar pattern in relation to two-lane, 24-foot bituminous pavements with 6- to 10-foot gravel shoulders. It appears therefore that paved shoulders adjacent to a two-lane, 22-foot pavement increase the effective surface width about 2 feet.

Vehicle speeds and lateral positions on the sections in Oregon with red and black combinations of pavements and shoulders were similar regardless of the order of color. Furthermore, there was no significant difference between traffic operations on sections with red and black combinations and those with contrasting shoulders and lanes of bituminous material.

Effectiveness of Edge Striping

Special studies of the effect of edge stripes on vehicle speeds and placements were conducted in conjunction with the studies of matching pavement and shoulder sections. These pavements appeared to be two 20-foot lanes without shoulders. Edge stripes were studied in cooperation with the State highway departments of Arizona, Utah, and Idaho. Standard centerline markings were used in all three States. These consisted of 4-inch-wide white reflectorized dashes, 15 feet long with 25-foot spaces, as illustrated in figure 8.

Two sections in Arizona, both of which were level tangents and on the same highway, were studied simultaneously. One section had no edge stripes; the other had 4-inch white reflectorized dashes, 7 feet long with 33-foot spaces. The dashes were 1 foot from the pavement edge or 19 feet from the centerline of the roadway.

In Utah, the studies were conducted at the same location before and after the edge stripes were painted. The edge stripes were 4-inch solid yellow reflectorized, and were placed 1.5 feet from the pavement edge or 18.5 feet from the centerline. Two locations were studied, one on a level tangent and the other near the top of a 3-percent grade about 2,000 feet long.

The speed data shown in table 8 indicate

Table 6.—Passing maneuvers observed on one-half mile sections of two-lane bituminous-surfaced rural highways with 12-foot traffic lanes and bituminous-paved shoulders

Traffic data	Shoulder surface types and widths		
	Bituminous and gravel shoulders, 8–10 feet [1]	Bituminous shoulders, 6–10 feet [2]	Bituminous shoulders, 8 feet [3]
Number of locations studied	3	11	13
Total hours of study	24	87	95
Average traffic volume vehicles per hour	210	210	120
Number of passings per half mile:			
Two vehicles abreast:			
Passing vehicle, right of centerline	------	------	4
Passing vehicle, straddling centerline	329	1,072	385
Passing vehicle, left of centerline	55	271	252
Three vehicles abreast [4]	2	57	8
Four vehicles abreast	------	2	2
Total passing maneuvers	386	1,402	651
Number of passings per mile per hour	32.2	32.2	13.7
Equivalent number of passings per mile per hour for—			
Traffic volumes of 210 vehicles per hour	32.2	32.2	[5] 42.0
Traffic volumes of 120 vehicles per hour	[5] 10.5	[5] 10.5	13.7

[1] Four feet of bituminous contrasting with traffic lane and 4 to 6 feet of gravel.
[2] Contrasting with traffic lane.
[3] No contrast with traffic lane.
[4] Two vehicles traveling in one direction.
[5] Computation based on the assumption that the number of passings varies as the square of the hourly traffic volume.

Table 7.—Summary of speeds and placements on two-lane portland cement concrete rural highways with 12-foot traffic lanes and 8- to 10-foot bituminous or gravel shoulders

Items of information	Bituminous shoulders	Gravel shoulders
Number of locations studied	3	3
Average traffic volume ... vehicles per hour	200	310
Number of vehicles studied	4, 928	5, 000
Average speed:		
Passenger cars ... miles per hour	51. 3	50. 9
Commercial vehicles ... do	45. 1	42. 6
Average lateral position: [1]		
Passenger cars:		
Free moving ... feet	6. 0	5. 2
Meeting other vehicles ... do	6. 7	5. 9
All other passenger cars ... do	6. 2	5. 4
Commercial vehicles:		
Free moving ... do	6. 2	5. 9
Meeting other vehicles ... do	6. 5	6. 6
All other commercial vehicles ... do	6. 2	6. 0
Vehicles straddling centerline:		
Passenger cars:		
Free moving ... percent	. 4	1. 6
Meeting other vehicles ... do	. 1	. 5
All other passenger cars ... do	. 6	1. 1
Commercial vehicles:		
Free moving ... do	. 6	-------
Meeting other vehicles ... do	-------	-------
All other commercial vehicles ... do	4. 1	1. 6
Vehicles encroaching on shoulder:		
Passenger cars:		
Free moving ... do	3. 6	-------
Meeting other vehicles ... do	5. 2	-------
All other passenger cars ... do	3. 5	(2)
Commercial vehicles:		
Free moving ... do	8. 5	1. 0
Meeting other vehicles ... do	10. 3	5. 0
All other commercial vehicles ... do	7. 3	1. 3
Clearance between bodies:		
Passenger cars meeting other vehicles ... feet	7. 1	5. 5
Commercial vehicles meeting other vehicles ... do	6. 3	5. 7

[1] Distance center of vehicle was from centerline of pavement.
[2] Less than 0.05 percent.

that edge stripes caused an increase in speeds in some cases and a decrease in speeds in others. The wide disparity in the results makes it impossible to draw any conclusion as to the effect of edge stripes on vehicle speeds. A study of the lateral positions of vehicles offers a much more conclusive measure of the effect of edge stripes, the results of which are also included in table 8.

The edge stripe used in Arizona caused drivers of passenger cars to travel during the day 0.1 foot closer to the centerline of the pavement than when there was no stripe, and at night they traveled 0.4 foot farther from the centerline. The edge stripe used in Utah had practically no effect on passenger car placements at night. During the day, however, the average lateral position was 0.6 foot farther from the centerline of the pavemen on the level tangent and 0.9 foot farther fron the centerline on a 3-percent downgrade. O the 3-percent upgrade, passenger cars travele 1.0 foot closer to the centerline of the pavement with the edge stripe than without it For daytime operations, the change in th average lateral positions of commercial vehicles caused by edge stripes in these tw States was about the same as the change fo passenger cars.

Along with the study of speeds and placements, an evaluation was made of the relatio of striping to shoulder encroachment. Because the pavement consisted, in effect, o two 20-foot lanes, vehicles were considered t be encroaching on the shoulder area when th right side extended more than 12 feet fro the centerline of the roadway. This encroachment is shown in the last four columns o table 8.

During the daytime, passenger cars observed in Arizona encroached on the shoulde about the same extent whether the pavemen edge was striped or not. At night, however a considerably larger percentage of cars use the shoulder as a result of edge striping. I Utah, the stripe on the level tangent sectio had very little effect on shoulder encroachmen of passenger cars traveling during the day o night. On the 3-percent downgrade, th stripe caused passenger cars to encroach o the shoulder more during the day and less a night. On the upgrade, the stripe reduce shoulder encroachment during the day an night.

As was mentioned earlier, nearly 80 percen of the trucks encroached on the shoulder are of matching shoulder and traffic lane sections

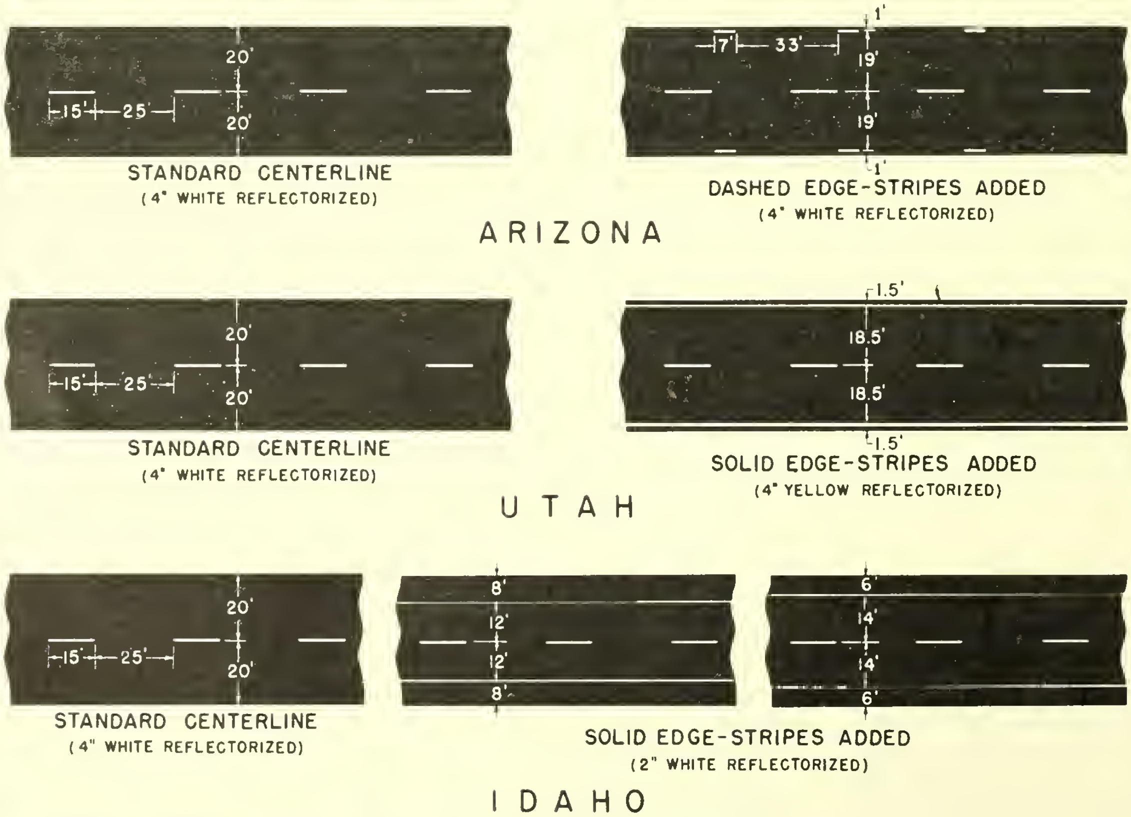


Figure 8.—Edge striping in three States on two-lane bituminous-paved rural highways with matching shoulders and traffic lanes.

Table 8.—Effect of edge striping on speeds and lateral positions of vehicles and their encroachment on shoulders of two-lane bituminous pavements with full-width paved shoulders having the same appearance as the traffic lanes

State	Gradient	Edge stripe				Change in speed resulting from edge striping [2]		Change in lateral position resulting from edge striping [3]		Encroachment on shoulder area [4]			
										Daytime		Nighttime	
		Color	Type	Width	Distance from edge [1]	Daytime	Nighttime	Daytime	Nighttime	Without stripe	With stripe	Without stripe	With stripe
PASSENGER CARS													
	Percent			*Inches*	*Feet*	*M. p. h.*	*M. p. h.*	*Feet*	*Feet*	*Percent*	*Percent*	*Percent*	*Percent*
Arizona	0	White	Dashed	4	1.0	+7.3	+9.5	−0.1	0.4	43.4	45.2	3.7	21.7
Utah	0	Yellow	Solid	4	1.5	+2.2	−2.5	.6	−.1	32.1	34.8	22.3	21.3
Do	−3.0	do	do	4	1.5	−3.9	+.9	.9	−.1	44.5	57.0	8.1	3.8
Do	+3.0	do	do	4	1.5	+4.7	+3.8	−1.0	−.1	45.7	30.8	11.4	8.9
Idaho	0	White	do	2	8.0	−9.2	−9.1	−1.9	−2.5	60.7	24.9	(5)	23.7
Do	0	do	do	2	6.0	−6.9	−4.7	−1.6	−2.3	60.7	31.0	(5)	25.9
COMMERCIAL VEHICLES													
Arizona	0	White	Dashed	4	1.0	+6.2	(5)	−0.2	0.1	97.6	98.5	100.0	66.7
Utah	0	Yellow	Solid	4	1.5	+2.3	−3.3	.5	.2	56.1	65.0	48.6	53.3
Do	−3.0	do	do	4	1.5	−2.3	−4.5	.9	1.3	46.0	74.2	6.2	35.0
Do	+3.0	do	do	4	1.5	+5.6	+7.0	−1.1	−.1	86.4	69.1	45.4	43.7
Idaho	0	White	do	2	8.0	−4.6	−4.2	−1.7	−2.0	86.8	42.9	(5)	46.4
Do	0	do	do	2	6.0	−1.4	0	−1.7	−2.6	86.8	61.0	(5)	51.3

[1] Distance toward centerline from edge of the full-width paving (12-foot traffic lane plus 8-foot shoulder, or a total of 20 feet).
[2] Average speed observed on striped section compared with that on unstriped section.
[3] Average lateral position on striped section compared with that on unstriped section. Minus values indicate that the average position on striped section was closer to the centerline of the highway than on the unstriped section.
[4] Percentages of vehicles with right side extending more than 12 feet from centerline of roadway.
[5] No data available.

In Arizona, edge stripes were not effective in reducing encroachment except at night. The tests made in Utah indicated that trucks made greater use of the shoulders after striping, except on the one section with the 3-percent upgrade.

The studies of edge striping in Idaho were conducted on one level tangent section of highway. The original 4-mile section was divided into two sections of equal length. Two-inch-wide solid reflectorized stripes were painted on one section 8 feet from the pavement edge or 12 feet from the centerline, as shown in figure 8. On the other section the stripe was painted 6 feet from the edge or 14 feet from the centerline.

Table 9.—Vehicle speeds related to special signs and edge stripes on a two-lane bituminous pavement in Oregon

Conditions of study [1]	Average speed	Vehicles traveling over 60 m. p. h.	Vehicles traveling under 40 m. p. h.
PASSENGER CARS			
Daytime:	*M. p. h.*	*Percent*	*Percent*
Normal conditions	61.3	60.0	1.8
Signs only	58.3	43.0	3.3
Signs and edge stripes	61.3	60.5	1.4
Edge stripes only	57.9	46.5	3.0
Nighttime (edge stripes only)	56.7	41.5	1.8
COMMERCIAL VEHICLES			
Daytime:			
Normal conditions	53.2	18.0	3.2
Signs only	51.3	13.0	5.6
Signs and edge stripes	52.7	13.2	1.3
Edge stripes only	49.6	5.4	3.9
Nighttime (edge stripes only)	49.0	6.3	1.4

[1] Legend on signs: NO TRAVELING ON PAVED SHOULDERS. Edge stripes were 4-inch solid yellow reflectorized, 13 feet from centerline of roadway.

No firm conclusion can be drawn as to the effect of these edge stripes on vehicle speeds, but vehicles traveled considerably closer to the centerline of the pavement as a result of striping. The effect was greater at night than during the day, and generally speaking, stripes placed 8 feet from the edge of the shoulder influenced drivers more than those placed at 6 feet. As a consequence of the tendency to crowd the centerline, there was less travel on the shoulders.

Although fewer vehicles in Idaho traveled during the day on the shoulder area of the section with stripes placed 8 feet from the shoulder edge as compared with those placed at 6 feet, some vehicles tended to use the 8 feet to the right of the stripe as a traffic lane. In effect, the pavement was used as a four-lane undivided highway except that the distribution of traffic between lanes was reversed. In other words, the percentage of vehicles traveling in the 12-foot lane approximated the percentage that normally would travel in the right lane of a four-lane undivided highway with an equivalent volume of traffic.

Special Studies in Oregon

A level tangent section of highway in Oregon was selected for a study of special edge stripes and signs. The section of highway consisted of two 12-foot bituminous lanes flanked by 10-foot shoulders—4 feet of contrasting bituminous material and 6 feet of gravel.

Speed and placement data were recorded at the same point on the highway under the conditions shown in table 9. The average speeds of passenger cars and commercial vehicles for the five conditions of study and the percentages of vehicles traveling above 60 miles per hour and below 40 miles per hour are included in this table. Under normal conditions, the average speeds were 61.3 miles per hour for passenger cars and 53.2 miles per hour for trucks. The lowest average speed during the daytime was observed when only the edge stripes were used, whereas the lowest percentage of passenger cars traveling over 60 miles per hour during the day was observed when only the signs were used. The per-

(Continued on page 215)

Table 10.—Lateral positions of vehicles related to special signs and edge stripes on a two-lane bituminous pavement in Oregon

Conditions of study [1]	Lateral position [2]	Vehicles encroaching on shoulders	Clearance between bodies of meeting vehicles
PASSENGER CARS			
Daytime:	*Feet*	*Percent*	*Feet*
Normal conditions	6.6	10.7	8.1
Signs only	6.4	8.4	7.9
Signs and edge stripes	6.1	4.3	7.2
Edge stripes only	6.1	5.4	7.0
Nighttime (edge stripes only)	6.0	3.7	7.6
COMMERCIAL VEHICLES			
Daytime:			
Normal conditions	7.2	41.3	8.4
Signs only	6.6	30.7	7.7
Signs and edge stripes	6.3	18.2	6.2
Edge stripes only	6.2	13.1	7.2
Nighttime (edge stripes only)	6.1	6.3	6.3

[1] Legend on signs: NO TRAVELING ON PAVED SHOULDERS. Edge stripes were 4-inch solid yellow reflectorized, 13 feet from centerline of roadway.
[2] Distance from center of vehicle to centerline of roadway.

The AE-55 Indicator Used for Determining Air Content of Concrete

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by WILLIAM E. GRIEB,
Highway Physical Research Engineer

THE AE-55 air indicator or Chace air meter is a pocket-sized device intended for use in the field to estimate the air content of plastic concrete. This apparatus was developed by L. M. Chace, a consulting engineer of North Bridgeton, Maine. A number of State highway departments and other organizations have purchased this device and are using it experimentally in the field. They are interested in it because of its low cost, rapidity of operation, and convenience to the engineer in the field.

Tests were made in the laboratory of the Bureau of Public Roads to obtain information on the accuracy and dependability of this apparatus. The air content of a large number of concrete mixes prepared in the laboratory was determined using this indicator, and the results were compared with those obtained by the gravimetric and the pressure methods, AASHO methods T 121 and T 152, respectively.

The AE-55 air indicator shown in figure 1 consists of two parts. One part is a small cylinder of Pyrex glass about 1 inch in diameter and 3 inches long which tapers at one end to a stem or tube about ¼ inch in diameter and 3 inches in length. This is similar to the filtration crucible holders shown in catalogs of laboratory apparatus. The other part is a rubber stopper with a brass cup mounted on the smaller end. The stopper fits the larger end of the glass cylinder. The brass cup is ¾ inch in diameter and ½ inch in depth, and has a volume of approximately 3.7 ml. When the stopper and cup are inserted into the cylinder, the volume of the latter is about 27 ml. Eleven equally spaced graduations are etched on the stem of the cylinder, each pair indicating a volume of about 0.08 ml.

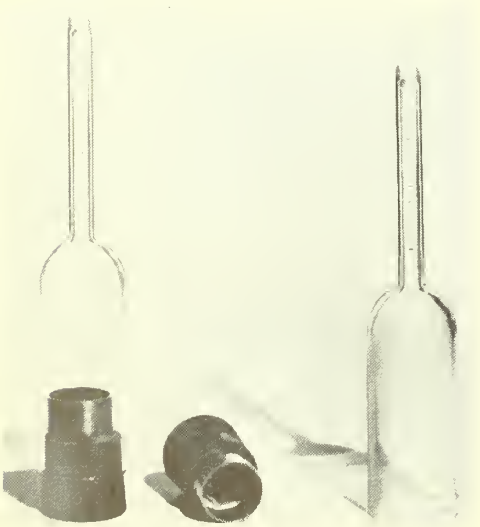


Figure 1.—The AE-55 indicator (assembled and unassembled) used to estimate air content in concrete.

When the concrete tested contains 15 cubic feet of mortar per cubic yard, each graduation on the stem of the indicator represents 1 percent of air in the concrete. If the concrete contains a different amount of mortar, the correction factors given in table 1 are applied. These factors were furnished by the manufacturer of the indicator.

Method of Test

In determining the air content of concrete with the indicator, the following procedure was used: The brass cup was filled with mortar from the concrete excluding particles of sand larger than about 1/10 inch. The mortar was compacted by rodding with a thin, stiff wire [2] and then struck off flush with the top of the cup. The sides of the cup and stopper were cleaned of mortar. The stem end of the cylinder was closed by holding the thumb over the end, and the cylinder was filled with denatured alcohol [3] to the mark on the cylinder. The stopper and cup were then inserted into the cylinder. The indicator was righted, the stopper pressed firmly into the cylinder, and the thumb was removed. The level of the alcohol was brought to the upper graduation on the stem of the cylinder by addition of alcohol or by slight movement of the stopper. When alcohol was added, a small syringe or dropper was used. Care was taken to remove all air bubbles from the cylinder and to keep the stopper seated firmly enough to prevent leaking of the alcohol.

The thumb was again placed over the open end of the stem and the indicator turned gently from a vertical to a horizontal position while the body of the cylinder was tapped with the palm of the hand. Care was taken not to disturb the setting of the stopper. This procedure was continued until all of the mortar had been dispersed into the alcohol and no more air bubbles appeared. As a result, the air in the mortar was replaced by alcohol. The indicator was then held in a vertical position and the new level of the alcohol read to the nearest half graduation on the graduated stem. The stopper was then removed and the indicator washed clean with water. Less than 3 minutes was required to make the test.

This method of determining the air is a volumetric method and is similar in principle to that described in ASTM Method C 173, "Air Content of Freshly Mixed Concrete by the Volumetric Method." In the ASTM method, the air in the fresh concrete is measured by displacing it with a liquid and then determining the volume of the liquid used. The ASTM method is not in common use except where the concrete contains slag or other porous aggregates.

Two other methods for the determination of air in plastic concrete are in general use. In ASTM Method C 231 (AASHO Method T 152), the air is determined by measuring the reduction in volume of the concrete when the latter is held in a closed container and subjected to a definite pressure. This method is used extensively and is considered the most reliable. In ASTM Method C 138 (AASHO Method T 121), the air is determined by calculation from the unit weight of the concrete and the batch weights and specific gravities of the materials used. The latter method is used where a pressure air meter is not available. Where specific gravities and

Table 1.—Conversion factors for the AE-55 air indicator used to correct the indicated air content when the concrete contains other than 15 cubic feet of mortar [1]

Mortar per cubic yard of concrete	Conversion factor [2]	Mortar per cubic yard of concrete	Conversion factor [2]
Cu. ft.		*Cu. ft.*	
10.0	0.67	15.5	1.04
10.5	.70	16.0	1.07
11.0	.73	16.5	1.10
11.5	.76	17.0	1.13
12.0	.80	17.5	1.16
12.5	.83	18.0	1.20
13.0	.86	18.5	1.23
13.5	.90	19.0	1.26
14.0	.93	19.5	1.30
14.5	.96	20.0	1.33
15.0	1.00	----------	--------

[1] Factors furnished by manufacturer of apparatus.
[2] Multiply reading on stem of indicator by conversion factor to obtain correct air content.

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[1] This article was presented at the 36th Annual Meeting of the Highway Research Board, Washington, D. C., January 1957.

[2] The wire in a No. 1 Gem paper clip is suitable.

[3] This alcohol was used instead of water to prevent foaming of the liquid when mortar was added.

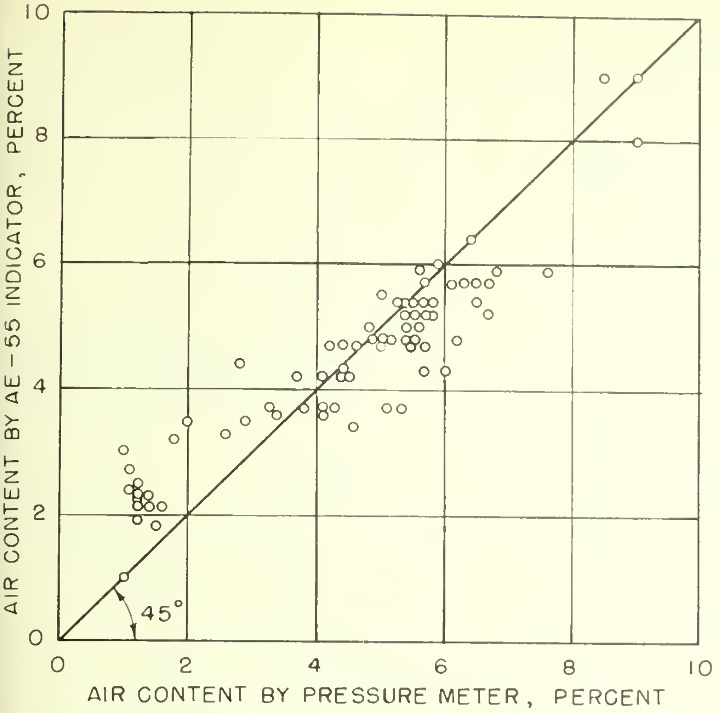

Figure 2.—Air content of concrete as determined by the AE–55 indicator and standard pressure meter.

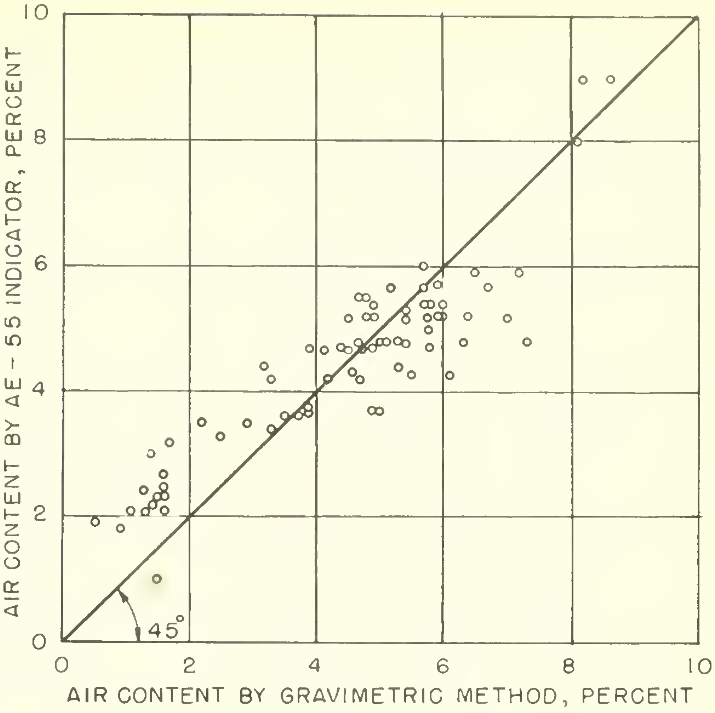

Figure 3.—Air content of concrete as determined by the AE–55 indicator and gravimetric method.

weights are correct and a representative sample is obtained, this method should give an accurate measure of the air content.

Comparison of Three Methods for Measuring Air Content

To determine the suitability of the AE–55 indicator, tests of 84 different concrete mixes were made using this indicator and the pressure and gravimetric methods. The concrete mixes were prepared using different cements and aggregates, and different amounts of air-entraining admixtures to give air contents varying from 1 to 9 percent as determined by the pressure method. Each value reported for the AE–55 indicator is an average of two tests usually made by two operators who generally found results agreeing within 0.5 percent of air. Each value determined by the pressure or gravimetric methods is for a single test. A comparison between the results obtained for each mix by the pressure method and the AE–55 indicator is shown in figure 2. A similar comparison between the results obtained by the AE–55 indicator and the gravimetric method is shown in figure 3.

Table 2.—Average values for different ranges in air content of concrete

Range in air content by pressure meter	Number of samples tested	Average air content measured by—			
		Pressure meter	Gravimetric method	AE–55 indicator	Difference, col. 3 minus col. 5
Pct.		*Pct.*	*Pct.*	*Pct.*	*Pct.*
1.0–1.9	14	1.28	1.36	2.26	−0.98
2.0–2.9	4	2.58	2.70	3.68	−1.10
3.0–3.9	4	3.55	3.65	3.80	−.25
4.0–4.9	14	4.41	4.55	4.24	.17
5.0–5.9	33	5.45	5.36	5.12	.33
6.0–6.9	10	6.42	6.24	5.38	1.04
7.0–7.9	1	7.6	6.5	5.9	1.7
8.0–8.9	4	[1] 8.75	8.28	8.75	.0

[1] Estimated air content—meter can only be read to 8.0 percent.

Average values for several different ranges in air content are given in table 2, together with the difference between the average values for the pressure meter and those for the AE–55 indicator. This table shows good concordance of the average air contents determined by the pressure and gravimetric methods for all values except those above 7 percent. However, the pressure air meter read only to 8 percent, and higher values shown for this meter were estimated. The data show that either method may be used with general assurance that the values obtained indicate accurately the amount of air in the concrete.

The results obtained with the AE–55 indicator did not show as good concordance with those obtained by the pressure meter. For values of air less than 3.0 percent as determined by the pressure meter, the AE–55

(*Continued on page 220*)

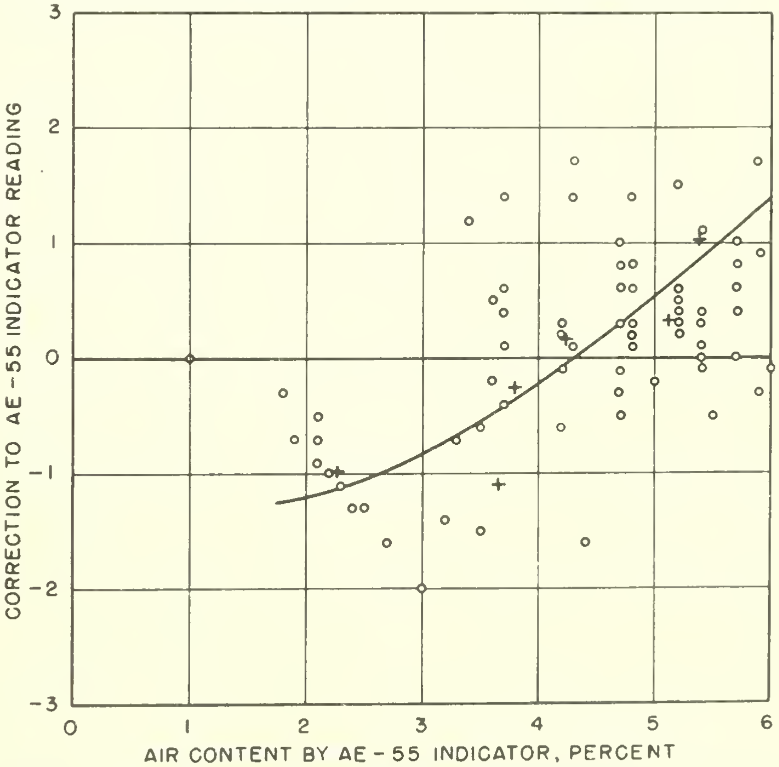

Figure 4.—Correction for AE–55 indicator reading to agree with air content determined by pressure meter.

A Study of Traffic Characteristics in Suburban Residential Areas

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by WILLIAM L. MERTZ, Highway Transport Research Engineer

Suburban land use involving the construction of garden-type apartments and large subdivisions of single homes imposes an enormous traffic burden on existing highway and street systems. In this article, traffic generation resulting from suburban residential development is discussed. Two typical housing facilities, located within the metropolitan area of Washington, D. C., were selected for study.

It was discovered that the two typical housing facilities added as much as 0.8 vehicle per dwelling unit per hour to the adjacent highways during the peak period of traffic flow. Whether the housing facility was a garden-type apartment development or a subdivision of single homes with an equivalent number of dwellings, the traffic generated per dwelling unit followed a similar pattern and volume.

IN RECENT YEARS there has been an exodus of city dwellers to the suburbs, a trend brought about by the widespread ownership of automobiles and the mass construction methods in housing. Emphasis in housing production has changed from the old way of dwellings being constructed one at a time by individual builders to the mass production of hundreds and even thousands of homes by one company. The merchandising of homes has evolved in the same direction as that of the family car. One may select today a mass-produced home that is similar to his neighbor's in the basic construction features and differing only in color, trim, and interior appointments.

The automobile has also drastically altered the environment of the apartment dweller. Apartment developments too have become larger and farther removed from the facilities of the central business district and work centers. No longer is the immediate availability of mass transit facilities the major factor in the location of such developments. These types of residential developments have in turn created problems in highway planning by introducing a change in land use and a concomitant change in travel requirements for the existing highway and street systems. In the past, estimates of future automobile owner-

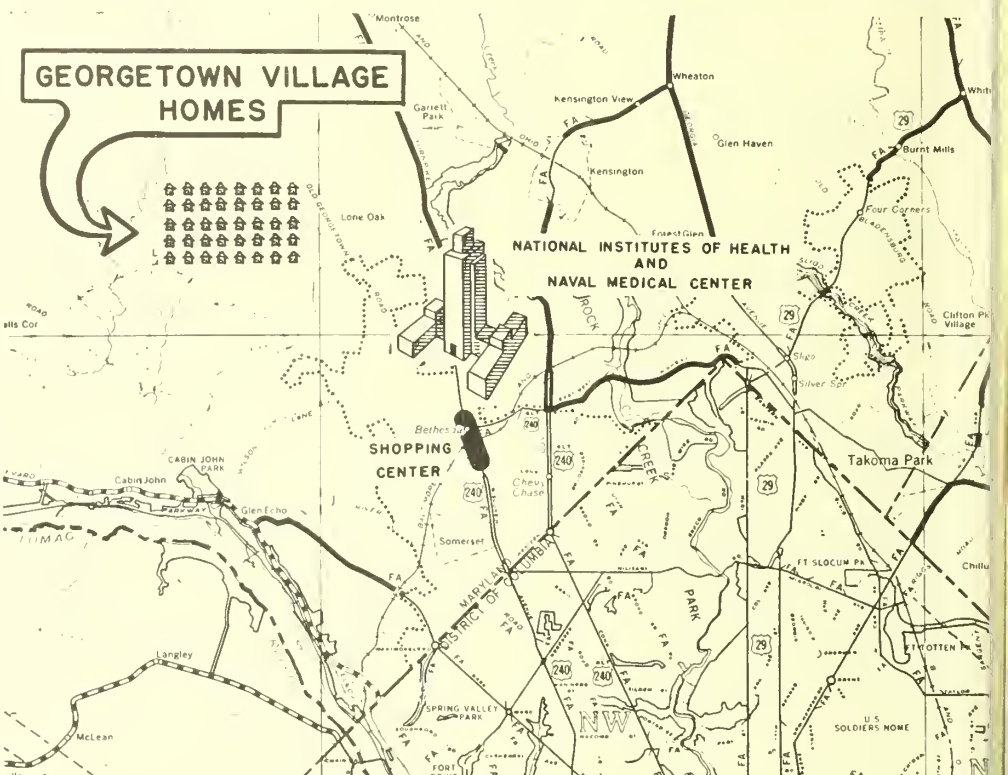

Figure 1.—Map showing the location of the two residential areas studied and their positions with respect to the central business district and other major traffic generating facilities.

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[1] The field work and part of the summarizing of the data were done by eight junior highway engineers of the Bureau of Public Roads as a phase of their in-service training in May 1955. Acknowledgment is given the following employees: Frank S. Allison, Daniel H. Brown, Charles M. Moffet, William S. Peterson, Robert A. Quist, Glade W. Roberts, Charles B. Totten, and Sheldon C. Turnidge.

ship and use have generally been too conservative. Consequently, new highway facilities have usually reached their capacity within a few years after construction. Automobile ownership has increased even faster than the increase in population, and the incidence of two-car families is increasing each year.

Factors to Consider in Urban Planning

The Bureau of Public Roads is engaged in continuing research in urban traffic problems. Its research in urban highway planning includes the development and testing of methods of forecasting highway needs. In order to better understand the many complex problems, it has been necessary to assemble much data regarding the travel patterns and habits of the residents of urban areas. Some of the most fruitful sources of information are the urban origin-and-destination traffic studies that have been conducted in most of the major cities. By the use of home interviews, a great wealth of reliable information has been obtained relative to where, why, and how often people make trips. It has been possible to determine the effect of income, population density, and automobile ownership on the number of average daily trips per dwelling unit.

The development of the origin and destination studies involving home interviews has, in recent years, provided engineers and planners with data to help solve today's highway problems, but to estimate future needs they must extrapolate current data. Estimating population growth is a well-developed science, but where this population will locate is not so clearly defined. Forecasting highway needs involves an estimation of future population trends, population density, types of housing, automobile ownership, and vehicle use. The planner must resort to estimates of future growth of urban areas based on some philosophy of expected changes in land use. If reasonably accurate land-use forecasts can be obtained and the traffic that will be generated by these changes can be estimated, the planning engineer is then in a position to apply the data developed by the home-interview type of traffic study to future trends.

Purpose of Study

This article is directed specifically to the feasibility of forecasting traffic demands based upon knowledge of the relation of traffic to residential land use in lieu of the more detailed origin-and-destination traffic studies. In other words, if the forecaster has information about the number of dwelling units per acre, existing and proposed, can he accurately predict the traffic generated by these areas? Also does the type of dwelling unit (apartment or individual home), its distance from the central business district, automobile ownership, and the availability of mass transit facilities appreciably affect traffic generation? Further, can the city planner or engineer predict the effect of changes or the granting of exceptions to the zoning laws on traffic generation?

Sites Selected

The apartment development and the individual home areas studied represent the two extremes of housing facilities in the major cities. Considerable effort was made to select two areas that have quite different characteristics but represent the same economic level. The apartment development has high population density, mass transit facilities available, adjacent shopping facilities, and is located 6 miles from the central business district. The individual home development has relatively low population density, limited transit facilities, no extensive shopping facilities in the immediate neighborhood, and is relatively distant, 12 miles, from the central business district. The location of the two residential areas and their positions with respect to the central business district can readily be ascertained from the map shown in figure 1.

Parkglen is a 216-unit garden-type apartment development located in a built-up area in Arlington County, Virginia. One hundred and ninety-five of the apartments were occupied at the time of the study. The monthly rental rate was $81.50 for a one-bedroom unit and $91.50 for two bedrooms.

The other residential area, Georgetown Village, is a group of 156 single-unit homes in the $13,000–$14,000 price range located at the periphery of the suburban area in Montgomery County, Maryland. One hundred and fifty homes were occupied at the time of the study. Since each of these housing areas was served by one access road, only a small force of observers was needed to record the movement of traffic. The Parkglen apartment development was studied Monday and Tuesday of the first week in May 1955, and Georgetown Village, Monday and Tuesday of the following week. Entering and departing traffic for both areas was observed continuously from 6 a. m. to 10 p. m. An automatic traffic counter was available at Parkglen to verify the recorders' tabulations.

An inventory of passenger cars parked in the areas and their license plates was made during the early morning hours before the studies began. These were assumed to be residents' vehicles. During the study, the recorders noted by hour periods, the State of registration and the license number of each vehicle that entered or left the area for identification as a resident or a nonresident vehicle. During the morning and evening peak-traffic volume periods, however, the information was obtained for quarter-hour intervals. In addition, other pertinent information was recorded such as number of passengers per car, number and type of service trucks, and the number of pedestrians.

Summary of Observations

On the basis of peak traffic volumes recorded during the highest 15-minute interval, it appears that the construction of residential properties similar to Parkglen apartments and Georgetown Village would add about 0.8 vehicle per dwelling unit per hour to the adjacent highway facilities. If the highest hourly volume were used instead of the highest 15-minute volume, the number would be reduced to 0.55 vehicle per dwelling unit per hour. In this study, the average daily number of trips for each of the two residential areas was about five trips per dwelling unit. Other studies have indicated that the traffic patterns of the residential areas considered in this article would not be applicable to densely populated downtown areas where automobile ownership is low and resident parking space is at a premium.

This study has shown that traffic generated per dwelling unit as a result of the construction of either garden-type apartments or single-unit homes follows a similar pattern and volume.

In suburban residential development, highway planning officials must consider the capacity of existing highway and street systems to determine whether there is adequate capacity to accommodate the additional traffic generated through new residential development. If present usage is near capacity, then highway plans must be adjusted accordingly to provide for the increased traffic volume. If additional research confirms the findings of this study, it will be possible to predict future traffic generation of suburban residential land development for a given density of dwelling units.

Automobile Ownership Ratios

Table 1 summarizes the information recorded in the two studies. The automobile ownership ratio of 0.97 per dwelling unit for the apartments and 1.17 for the individual homes follows the same pattern that has been established by current studies based on origin and destination surveys; that is, automobile ownership increases with the distance from the central business district and decreases with a

Table 1.—Vehicular and pedestrian trips recorded for a 2-day period during May 1955

Traffic components	Parkglen apartments (195 occupied dwelling units; 190 resident passenger cars)			Georgetown Village homes (150 occupied dwelling units; 175 resident passenger cars)		
	Number of trips	Percent of total vehicular trips	Average number of trips per dwelling unit per day	Number of trips	Percent of total vehicular trips	Average number of trips per dwelling unit per day
Resident passenger cars	1,136	57.4	2.91	885	59.1	2.95
Nonresident passenger cars	657	33.2	1.68	483	32.3	1.61
Total, passenger cars	1,793	90.6	4.59	1,368	91.4	4.56
Service trucks	186	9.4	.48	128	8.6	.43
Total vehicles	1,979	100.0	5.07	1,496	100.0	4.99
Pedestrian	507	----------	1.30	93	----------	.31
Total	2,486	----------	6.37	1,589	----------	5.30

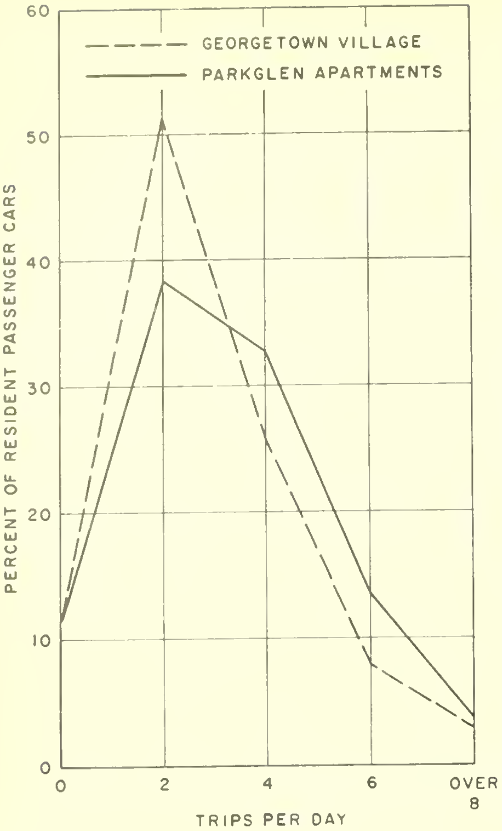

Figure 2.—Frequency of resident passenger-car trips observed at both residential areas.

rise in population density. Information derived from the 1948 Washington, D. C., origin and destination studies shows that average car ownership may be as low as 0.2 vehicle per dwelling unit in densely populated downtown residential areas and as high as 1.7 in some other residential areas. It is assumed that one of the major factors in such wide variation in automobile ownership in urban areas is the availability of parking facilities contiguous to the residence.

Such developments as the Parkglen apartments almost universally provide adequate parking for tenants, so availability of residential parking space is not considered to be a factor in automobile ownership in either Parkglen or Georgetown Village. However, it should be pointed out that there possibly were some two-car families in the Parkglen apartment area. There were six pairs of automobiles with consecutively numbered license plates; therefore, there may have been more apartment families without automobiles than the ratio of 0.97 automobile per dwelling unit might imply. On the other hand, there was at least one car for every dwelling unit at Georgetown Village, so even though the numerical difference in average car ownership between the two areas is not great, it is nevertheless a real and significant difference. Thirty percent of the tenants' vehicles at Parkglen apartments were registered in a State other than Virginia, whereas only 5 percent were registered "out-of-State" at Georgetown Village. This difference is attributed to the large turnover of transient population in apartments as compared with individual homes where it is assumed that most, if not all, are resident owned.

In Parkglen apartments, the outbound and inbound vehicle trips between the hours of 6 a. m. and 10 p. m. during the 2 days of study totaled 1,979. Of this total, 57.4 percent were resident trips and 33.2 percent were nonresident; the remaining trips, 9.4 percent, were made by service vehicles such as milk, bread, laundry and dry-cleaning trucks, and school buses. Excluding pedestrian trips, there were 2.91 resident trips per dwelling unit per day; 1.68, nonresident; and 0.48, service-truck trips.

In Georgetown Village, there was a total of 1,496 vehicle trips for the 2-day period: 59.1 percent, resident; 32.3 percent, nonresident; and 8.6 percent, service-truck trips. Again omitting pedestrian trips, there were 2.95, resident; 1.61, nonresident; and 0.43, truck trips. A comparison of the total vehicle trips per dwelling unit for the two areas shows remarkably close agreement—5.07 and 4.99 for Parkglen apartments and Georgetown Village, respectively. The 1948 origin-and-destination traffic study showed an average of 5.5 passenger-car trips per dwelling for the Washington, D. C., metropolitan area which agrees reasonably well with the data developed here.

Parkglen apartments had four times more pedestrian trips than Georgetown Village, 1.30 and 0.31 trips per dwelling unit per day. This difference is attributed to the availability of mass transit and rather extensive shopping facilities within walking distance of Parkglen apartments. Georgetown Village is not served directly by transit, and shopping facilities are over 3 miles distant. Approximately 40 percent of the pedestrians at Parkglen apartments were mass transit users.

This study does not reflect the total move-

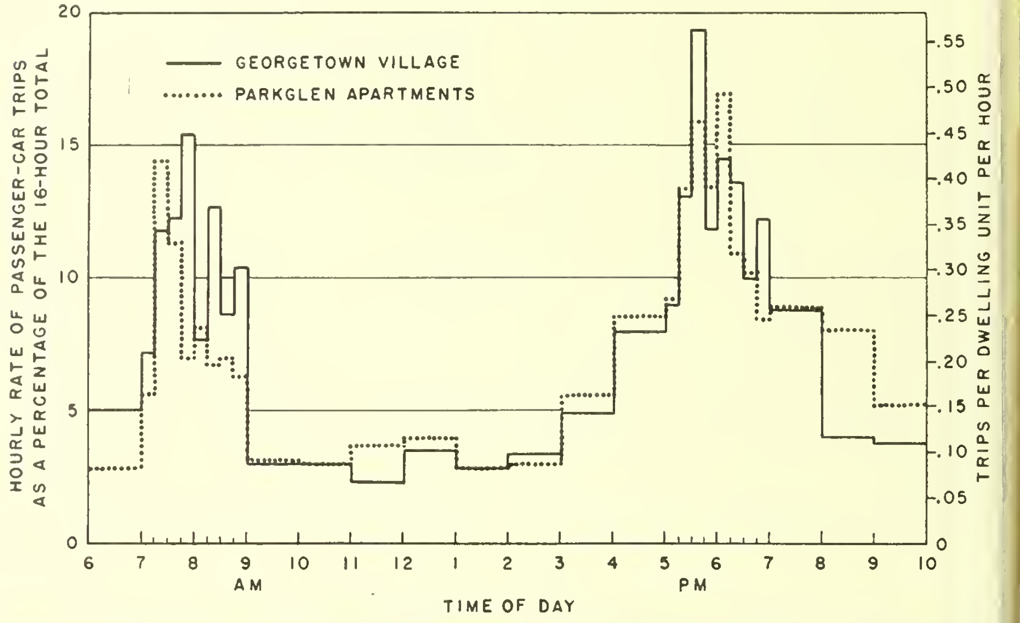

Figure 3.—Hourly rate of resident passenger-car trips with peak hourly rates plotted in 15-minute intervals for both residential areas.

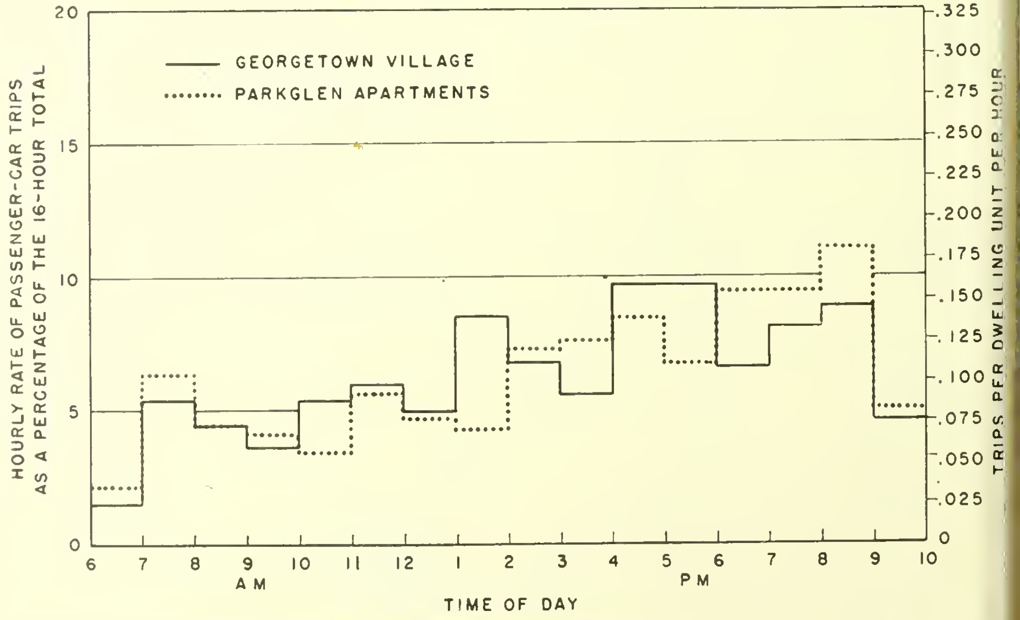

Figure 4.—Hourly rate of non-resident passenger-car trips for both residential areas.

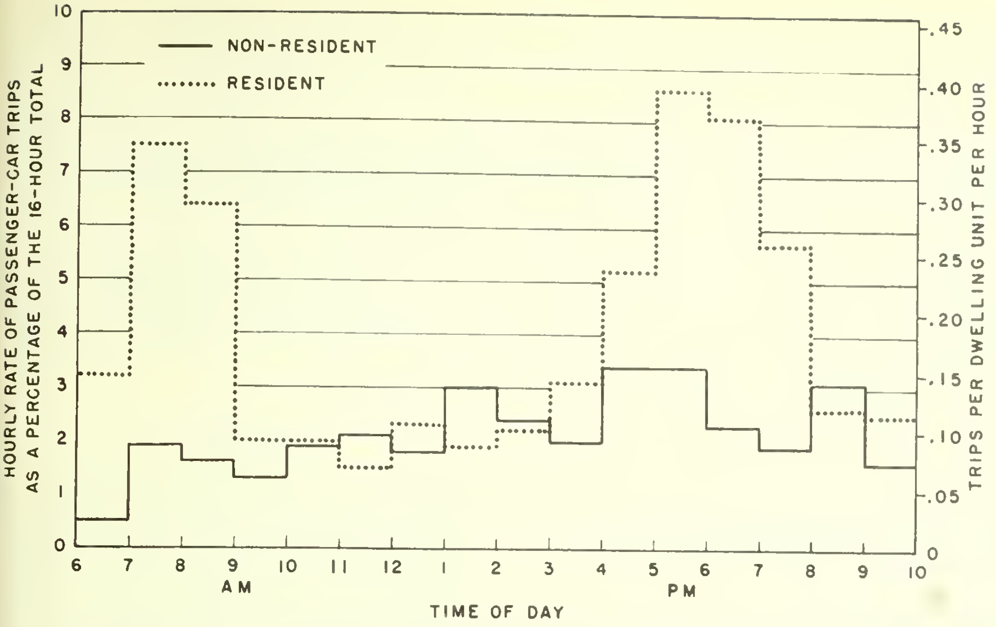

Figure 5.—Hourly rate of resident and non-resident passenger-car trips observed at Georgetown Village.

ments of residents in as much detail as the home-interview origin-and-destination traffic studies. It is believed that trips made by people living in comparatively isolated suburban areas such as Georgetown Village serve more than one purpose, whereas the apartment dwellers who are close to most of their shopping needs make more single-purpose trips. In the home-interview type study, the many purposes of trips are recorded; for example, a journey beginning and ending at home may include stops for a variety of purposes, each of which would be recorded as a separate trip. In the present study, only the movement of traffic to and from the two housing developments was tabulated.

Figure 2 shows the resident passenger-car trip frequencies. In both residential areas, about 12 percent of the resident automobiles made no trips at all outside the area on either day studied. About 51 percent of the automobiles in Georgetown Village, made 2 trips per day, whereas only 38 percent of the resident automobiles at Parkglen apartments made that number. However, a greater number of vehicles at the latter location made three or more trips per day than at the more distant Georgetown Village. This variation is explained by the probability that the more isolated suburbanite at Georgetown Village, though making less frequent trips, actually makes more multipurpose trips than the relatively close-in resident at Parkglen apartments.

As previously mentioned, residents of Georgetown Village owned more cars per dwelling unit than residents of Parkglen apartments and the average number of vehicle trips per day was almost identical for both areas, which indicates that trip frequency does not depend upon automobile ownership in this case. Car occupancy averaged 1.55 persons per car for the Parkglen apartment group, and 1.60 persons per car for Georgetown Village.

Figures 3–8 show the hourly rates of vehicular trips per dwelling unit for the various components of the traffic. It was possible to use the same trips per hour scale to represent both housing developments because, as indicated in table 1, the total trips per day per dwelling unit were almost identical for both areas for each of the components as well as the total vehicular traffic.

Peak Traffic Periods

Figure 3 shows the hourly pattern of resident vehicle trips for both study areas and the peak hourly rates plotted in 15-minute intervals. The morning peak-traffic volumes between 7 and 9 a. m. constitute about 20 percent of the total resident traffic for both study areas. The evening peak period begins to develop about 3 p. m. and does not subside until 8 or 9 p. m., and constitutes approximately 50 percent of the total resident traffic. If it is assumed that all of the morning peak flow (7 to 9 a. m.) is "to work" traffic, an equivalent 20 percent "from work" deducted from the evening peak period leaves 30 percent to be accounted for by residents making additional trips for shopping, recreation, and other purposes.

The California Division of Highways has published a report [2] of a 1947–48 origin-and-destination traffic survey for the city of Sacramento in which is presented a chart (p. 26) of automobile trips similar to figure 3. The morning peak traffic load is about 17 percent of the total and occurs between 7 and 9 a. m. The off-peak volumes are about 5 percent per hour and the evening peak volume, occurring between 3 and 8 p. m., is about 45 percent of the total. The study covered a period of 6 months and involved 328,000 trips.

Figure 3 shows that the elapsed time interval between the Georgetown Village morning and evening peak 15-minute periods is less by about 1 hour than that for Parkglen apartments, 9¾ and 10¾ hours, respectively. Apparently the average resident of Georgetown Village spends less time away from home during the working day than residents of Parkglen apartments which is only half the distance, 6 miles, from the central business district. It is probable that a number of Georgetown Village residents are employed near their homes, thus accounting for the shorter travel period. The National Institutes of Health and the Naval Medical Center, employing several thousand workers, are located about 2 miles from Georgetown Village. These Federal agencies are identified in figure 1.

The nonresident passenger-car traffic pattern is shown in figure 4. Fifteen-minute intervals of traffic movements during the morning and evening peak volumes were not

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[2] *Traffic Survey of the Sacramento Area.* State of California, Department of Public Works, Division of Highways, District III. 1947-48.

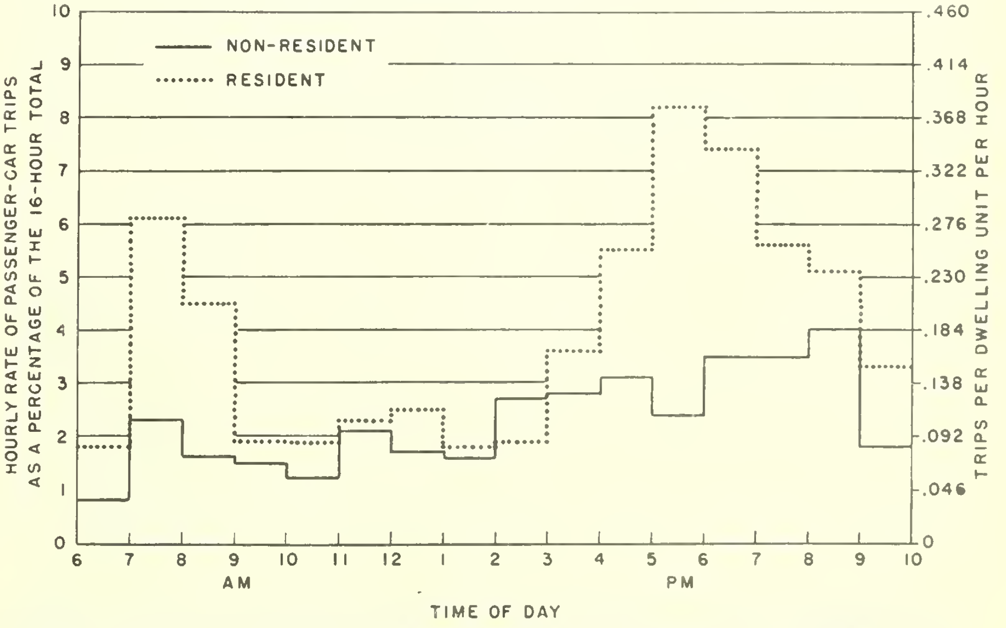

Figure 6.—Hourly rate of resident and non-resident passenger-car trips observed at Parkglen apartments.

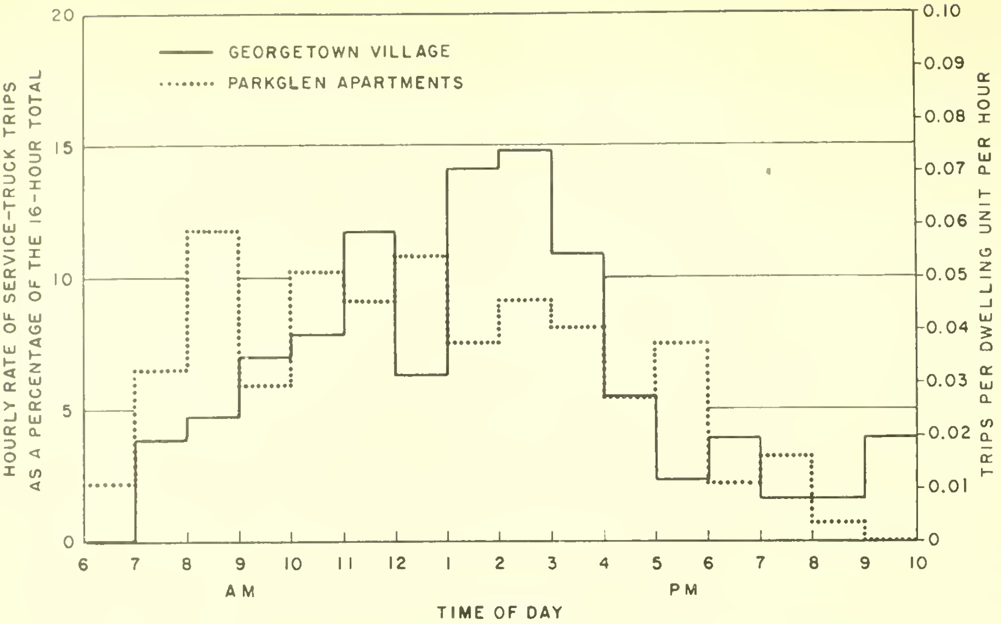

Figure 7.—Hourly rate of service-truck trips for both residential areas.

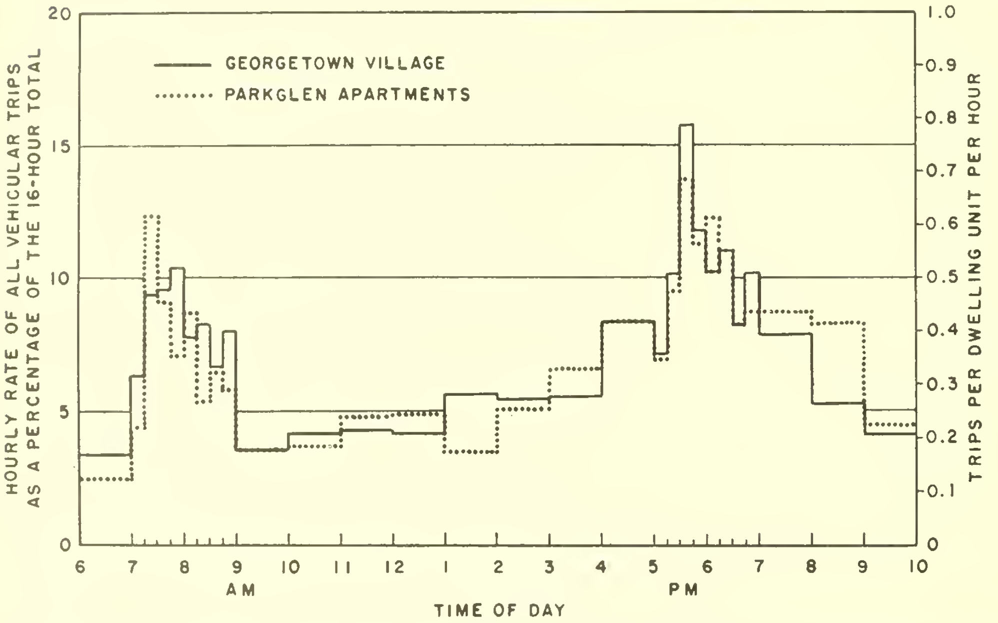

Figure 8.—Hourly rate of all vehicular trips with peak hourly rates plotted in 15-minute intervals for both residential areas.

plotted because little fluctuation was evident. The initial traffic load between 7 and 8 a. m. for both residential areas, particularly at Parkglen apartments, may be due to nonresident carpool drivers arriving to pick up passengers. The volumes then rise slowly throughout the day to a peak of about 0.18 vehicle per hour per dwelling unit at 8 p. m.

Figures 5 and 6 compare resident and nonresident traffic for each of the two areas during each hour of the day. It is significant to note that there is little variation between resident and nonresident traffic during the off-peak hours.

Service-Truck Trips Create No Problem

Service-truck trips to the two residential areas vary considerably in their hourly distribution, as shown in figure 7. Such trips to the apartment development are heaviest in the forenoon and, in general, decline steadily throughout the afternoon. Truck trips to Georgetown Village, however, reached their peak between 1 and 3 o'clock in the afternoon. This dissimilarity may be attributed to the distances involved with respect to the location of the central business district, suburban businesses, and shopping centers. It seems that delivery trucks serve the closer-in areas such as Parkglen earlier in the day. The peak volume of deliveries at Georgetown Village is greater and concentrated in a shorter period of time. The volumes are not particularly significant since peak service-truck travel does not occur at critical traffic flow periods.

All vehicular traffic is combined in figure 8 showing the trip volumes by hour periods and by 15-minute intervals during the hours of peak flow. The initial peak occurring between 7 and 9 a. m. accounts for about 15 percent of the total traffic. From a low point between 9 and 10 a. m., there is a gradual increase until the evening peak between 5 and 6 p. m. This upsurge in the afternoon traffic begins to develop about 2 p. m. and does not subside until after 9 p. m. This period accounts for about 60 percent of all trips. The greatest hourly variation between the two residential areas is about 3 percent but, in general, the traffic patterns of the two areas are quite similar.

The Behavior of Red Lead-Iron Oxide Primers When Exposed Directly to Weathering

BY THE DIVISION OF PHYSICAL RESEARCH BUREAU OF PUBLIC ROADS

Reported by WOODROW J. HALSTEAD,
Chemist

For special applications where considerable handling of the steel and some delay in applying the field coats of paint is anticipated, the Bureau of Public Roads has obtained good service with a red lead-iron oxide type shop coat primer. This paint is based on a pigment formulation of red lead, iron oxide, and siliceous matter, as in the AASHO Specification M 72, Type III, and a vehicle containing a ratio of three parts of linseed oil to one part of alkyd resin solids together with the necessary thinners and driers.

Direct exposure tests up to 6 years indicate that primers containing this type pigment and ratios of linseed oil to alkyd resin solids from 1:1 to 3:1 all had excellent weather resistance. Primers with a ratio of 5:1 showed some failure at 6 years. Unprotected straight red lead-linseed oil panels failed at 3 years.

DURING the reconstruction of the Philippine Island bridges after World War II, a program carried out under the direction of the Bureau of Public Roads, some difficulties with the shop coat of paint on the structural steel were experienced. Since the steel for a number of these bridges was fabricated in this country, shipment of the material to the job site required considerable handling. As a result, the typically soft film of the straight red lead-linseed oil primer conforming to the American Association of State Highway Officials (AASHO) Specification M 72, Type I, was often mechanically damaged to the extent that proper protection to the steel was not provided.

After a study of the problem, it was recommended that the shop coat be changed to a red lead-iron oxide type similar to that specified in AASHO M 72, Type III, except that the vehicle should contain a ratio of three parts of raw linseed oil to one part of alkyd resin solids instead of the 1:1 ratio in the AASHO specification. The full text of the modified specification is shown in the appendix.

This change was based on considerations of the need for the better wetting characteristics supplied by the raw linseed oil and the need for a paint that dried to a harder film than one containing linseed oil as the drying oil in the vehicle. The 3:1 ratio was chosen after an examination of the drying and visual characteristics of films applied with paints containing various ratios of linseed oil to alkyd resin solids ranging from 1:1 to 5:1. Reports from the field indicated that the adoption of the revised specification successfully eliminated the major portion of the damage to the paint film, and no indication of poor adhesion was apparent.

Since steel shipped overseas or to South American countries may be exposed to severe weathering conditions for relatively long periods of time before the shop coat is covered, the durability of such paints when exposed directly to weathering is of interest. Consequently, a limited investigation of this factor was begun at the same time that the red lead-iron oxide type paint was adopted for use on the Philippine Island bridges. This article reports the results of that investigation.

Procedure and Results

Table 1 shows the properties of seven paints used for exposure panels. Six of these were red lead-iron oxide types while the seventh was a red lead-linseed oil type. As shown, the red lead-iron oxide paints had ratios of linseed oil to alkyd resin solids ranging from 1:1 to 5:1. The alkyd resins were of two types, a long oil linseed-modified resin exemplified by Type II of Federal Specification TT–R–266[1] and a medium oil linseed-soya-modified as exemplified by Type III of that specification.

Each paint was applied to four panels. The first was a panel that had been allowed to rust and then the rust was mechanically cleaned from one-half of the surface. The remaining three panels were clean and free of rust. One of these was coated to give about the thickness that would be obtained with normal brush application. A second was coated with a thicker than usual film and the third was given two coats, the second being applied 24 hours after the first. All of these panels were then placed on an exposure rack outside the laboratory at an an angle of 45 degrees, facing south. The laboratory is located in Arlington, Va., and therefore the climate is typically that of the Middle Atlantic States. The area is essentially rural, and no industrial gases or salt spray are encountered.

Table 2 shows the average wet film thickness of each coating calculated from the actual weight of paint applied, the measured dry film thickness after 6 years exposure, and the rating of each panel at 1, 3, and 6 years based on a 0–10 point system. The assigned ratings between 4 and 8 were made to be as nearly comparable as possible to the

Table 1.—Properties of primers used in exposure tests

Characteristics of paint	Paint identification No.						
	1	2	3	4	5	6	7
Ratio of linseed oil to alkyd resin solids	3:1	3:1	2:1	2:1	5:1	1:1	(1)
Type of resin (TT–R–266)[2]	III	II	III	II	III	III	------
Weight per gallon........pounds	17.3	17.4	17.5	17.6	18.6	17.4	25.4
Pigment........percent	67.3	67.5	67.8	67.9	71.2	67.6	77.9
Vehicle........do	32.7	32.5	32.2	32.1	28.8	32.3	22.1
Drying time:							
Set to touch........hours	3½	4	3½	3½	7	1¼	------
Dry through within........do	24	24	24	24	24	16	24
Analysis of pigment:							
True red lead........percent	61.2	61.7	60.6	61.1	62.5	61.7	98.4
Iron oxide (Fe_2O_3)........do	12.5	12.8	12.4	12.7	14.0	12.0	0
Siliceous matter........do	18.7	18.7	18.8	18.8	17.6	17.8	0
Analysis of vehicle:							
Nonvolatile........do	66.2	68.1	66.3	66.9	69.8	57.7	87.4
Phthalic anhydride[3]........do	7.3	5.2	9.1	7.1	3.6	15.2	0
Oil acids[3]........do	78.0	84.4	74.9	77.9	79.2	71.1	------
Iodine number of fatty acids	------	------	------	------	------	------	184

(1) All linseed oil.
[2] Federal Specification TT–R–266, Type II, is a long oil linseed-modified resin; Type III is a medium oil linseed-soya modified.
[3] On basis of nonvolatile vehicle.

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[1] Copies of individual Federal Specifications may be purchased from the Superintendent of Documents, Government Printing Office, Washington 25, D. C.

Table 2.—Data for application and durability of exposure panels

Panel identification No.	Surface application conditions	Paint identification No.	Film thickness		Ratings [3] after exposure for—		
			Wet [1]	Dry [2]	1 year	3 years	6 years
			Mil.	*Mil.*			
S-1	Half clean-half rust	1	3.5	1.2	10	10	9
1	Clean-normal	1	2.3	1.0	10	10	10
2	Clean-thick	1	4.5	2.0	10	10	10
3	Clean-two coats	1	4.9	2.5	10	10	10
S-2	Half clean-half rust	2	3.1	2.0	10	10	7
4	Clean-normal	2	2.0	.8	10	10	8
5	Clean-thick	2	3.8	1.2	10	10	10
6	Clean-two coats	2	4.8	2.5	10	10	10
S-3	Half clean-half rust	3	3.6	2.0	10	10	8
7	Clean-normal	3	2.5	1.3	10	10	10
8	Clean-thick	3	5.2	2.0	10	10	10
9	Clean-two coats	3	5.4	2.5	10	10	10
S-4	Half clean-half rust	4	3.3	1.5	10	10	8
10	Clean-normal	4	2.5	1.0	10	10	10
11	Clean-thick	4	4.9	2.0	10	10	10
12	Clean-two coats	4	5.3	3.0	10	10	10
S-5	Half clean-half rust	5	3.5	1.5	10	9	6
13	Clean-normal	5	2.5	1.0	10	9	5
14	Clean-thick	5	3.6	1.0	10	9	6
15	Clean-two coats	5	4.7	2.0	10	10	9
S-6	Half clean-half rust	6	4.8	2.0	10	9	7
16	Clean-normal	6	3.4	1.5	10	10	10
17	Clean-thick	6	6.1	3.5	10	10	10
18	Clean-two coats	6	7.6	5.0	10	10	10
RL-1	Clean-normal	7	------	------	10	4	0
RL-2	Clean-normal	7	------	------	10	4	0

[1] Calculated from actual weight of paint applied.
[2] Measured after 6 years.
[3] Ten indicates no deterioration; zero indicates complete removal of film.

ratings that would be assigned under the "Standard Method of Evaluating Degree of Resistance to Rusting Obtained with Paint on Iron or Steel Surfaces," American Society for Testing Materials, Designation D 610–43. Expressed in a general way, it may be considered that panels rated 8 or above are in satisfactory condition and further service would be expected. Panels rated 7 show some signs of deterioration but only a very small amount of scraping and spotting would be necessary before recoating. A rating of 6 or 5 indicates that the paint is no longer adequately protecting the metal. Considerable scraping to remove rust would be necessary before recoating. A rating of 4 or lower indicates complete failure. Complete repriming after removal of rust and old paint would be required.

Discussion of Results

In any consideration of the results obtained from this series of exposure panels, the limited objective of observing the weathering properties of the primers themselves must be kept in mind. The results do not apply to complete paint systems in which the primer is adequately protected by intermediate and top coats. Although the unprotected red lead-linseed oil films were not greatly resistant to the deterioration from weathering forces, it should be noted that the chemical protection provided by this primer was such that no extensive rusting of the steel occurred until the red lead was completely removed from the surface. At 3 years, for example, most of the paint had been removed but in those areas where only a small amount of red lead remained, no rusting had occurred.

The most significant result of this investigation is the excellent weathering resistance of the red lead-iron oxide type primers containing ratios of linseed oil to alkyd resin solids not greater than 3:1. In all cases, primers containing 1:1, 2:1, and 3:1 ratios with either Type II or Type III resins have given excellent protection. No failure is apparent after 6 years except for some indication that films applied over rust—a practice that is definitely not acceptable in field applications—had become sufficiently permeable to moisture that rusting beneath the film had increased. The paint containing a 5:1 ratio of linseed oil to alkyd resin solids began to show some deterioration at about 3 years. At 6 years all of the one-coat applications had failed, although the two-coat application still retained a rating of 9.

The inhibitive action of these primers appeared to be adequate. For films applied over clean metal, there was no indication of failure at the very edge of the panels even though the edges and under surfaces were not coated and showed considerable rusting.

These experiments did not show any failure in adhesion over tight rust. However, since no mill scale or flaked rust was present on these panels, the apparently good adhesion obtained cannot be considered as being indicative of satisfactory adhesion to structural steel for all the combinations tested.

The excellence of straight red lead-linseed oil primers is established by years of satisfactory service, and therefore the superior weathering resistance and harder films obtained by the use of red lead-iron oxide primers may be of little consequence in many applications. For special cases where early and multiple handlings of the steel are necessary or where exposure to severe weathering conditions for a considerable period of time is likely, the red lead-iron oxide primers containing alkyd resin as part of the vehicle are recommended as being more satisfactory for use as shop coats than straight red lead-linseed oil type primers.

Although direct laboratory evidence was not obtained in this investigation, the general behavior of the paints examined and the reports obtained from field applications indicate that a ratio of one part alkyd resin solids to three parts of raw linseed oil is sufficient to provide a film of the desired hardening and drying characteristics without greatly affecting its wetting properties. In this respect, a formulation such as provided for in the Bureau of Public Roads specification shown in the appendix is considered more desirable than paint made as specified in AASHO Specification M 72, Type III.

Appendix

Specifications for red lead-iron oxide paint, used by the Bureau of Public Roads, are as follows:

(1) Pigment.—The pigment shall be as specified in table I. The iron oxide pigment shall be on a siliceous base (not calcium sulfate). The extracted pigment on analysis shall conform to the quantitative requirements given in table I under the subhead "Extracted pigment."

(2) Vehicle.—The vehicle shall consist of raw linseed oil blended with a glyceryl phthalate type varnish composed of a linseed oil modified resin together with the necessary driers and volatile thinners. The requirements for the vehicle are also shown in table I. The raw linseed oil shall conform to the Federal Specification TT–O–369. The alkyd resin shall conform to Federal Specification TT–R–266, Type 3. All vehicles shall be free from rosin and rosin derivatives. (The test for rosin subsequently shown under paragraph 11 shall be negative.) The vehicles may contain additional agents such as antioxidents and wetting aids.

(3) Paint quantitative requirements.—The paint shall meet the quantitative requirements shown in table II.

(4) Condition in container.—The paint shall not show excessive settling in a freshly opened full can, and shall easily be redispersed with a paddle to a smooth homogeneous state. The paint shall show no curdling, livering, caking, or color separation and shall be free from lumps and skins.

(5) Dilution stability.—The paint shall remain stable and uniform after reduction of eight parts by volume of the packaged material with one part by volume of mineral spirits conforming to Federal Specification TT–T–291, grade 1.

(6) Brushing properties.—The paint as received shall brush easily, possess good leveling properties, and show no running or sagging tendencies when applied at a spreading rate

of 500 square feet per gallon to smooth steel vertical surfaces.

(*7*) *Skinning.*—The paint shall not skin within 48 hours in a three-quarters filled closed container.

(*8*) *Appearance.*—The paint shall dry to a smooth uniform finish free from roughness, grit, unevenness and other surface imperfections. The paint shall show no streaking or separation when flowed on clean glass.

(*9*) *Sampling.*—Sampling shall be performed in accordance with Method 102.1 of Federal Specification TT–P–141.

(*10*) *Storage stability.*—The paint shall show no thickening, curdling, gelling, or hard caking when stored for 6 months from date of delivery in a full, tightly covered container at a temperature of 21 to 32 degrees centigrade (70–90° F.).

(*11*) *Testing.*—The paint shall be tested in accordance with applicable methods of Federal Specification TT–P–141 and as hereinafter specified. The following tests shall be conducted in accordance with that specification:

Test	*Method*
Percentage of pigment	402.1
True red lead (Pb_3O_4)	707.1
Iron oxide (Fe_2O_3)	714.1
Isolation of vehicle	403.2
Nonvolatile in vehicle (see note a)	405.3
Phthalic anhydride (procedure A)	702.1
Oil acids	703.1
Uncombined water	408.1
Consistency (Krebs-Stormer)	428.1
Coarse particles and skins	409.1
Weight per gallon	401.1
Set to touch time	406.1
Condition in container	301.1
Brushing properties	205.1, 432.1
Spraying properties	204.1, 433.1
Skinning	414.1
Rosin and rosin derivatives (see note b)	503.1
Flexibility (see note c)	622.1
Flash point	429.3
Storage stability	414.2

(*a*) *Method 405.3.*—A gravity convection oven may be used to determine the nonvolatile content of the supercentrifuged vehicle if the procedure outlined in Method 404.1 is modified as follows: Weigh accurately from 0.8 to 1.2 grams of sample (by difference), heat for 1 hour, cool, and weigh. Use the lower value to calculate the percentage of nonvolatile matter.

Table I.—Pigment and vehicle requirements

Ingredients	Requirements, percent by weight	
	Minimum	Maximum
PIGMENT		
Red lead (TT–R–191 Type I, grade C)	65.0	--------
Aluminum stearate	.3	0.4
Red iron oxide pigment (85 percent Fe_2O_3)	15.0	--------
Magnesium silicate	--------	14.7
Mica, 325 mesh	4.0	6.0
EXTRACTED PIGMENT		
True red lead, Pb_3O_4	62.5	--------
Iron oxide, Fe_2O_3	12.5	--------
Siliceous matter	--------	22.0
VEHICLE		
Raw linseed oil	49	--------
Alkyd resin solids	16	--------
Volatile thinner and drier	--------	35

Table II.—Quantitative requirements of paint

Characteristics	Quantitative requirements	
	Minimum	Maximum
Pigment, percent by weight	68	--------
Nonvolatile vehicle, percent by weight of vehicle	65	------
Phthalic anhydride, percent by weight of nonvolatile vehicle	7	--------
Oil acids, percent by weight of nonvolatile vehicle	78	--------
Uncombined water, percent by weight of paint	------	0.5
Coarse particles and skins (retained on No. 325 sieve), percent by weight of pigment	--------	2.0
Consistency (Krebs-Stormer), shearing rate at 200 r. p. m.:		
Grams	155	225
Equivalent KU	73	86
Weight per gallon pounds	17.0	--------
Drying time:		
Set to touch hours	--------	7
Dry through hours	--------	24
Flash point(degrees, Fahrenheit)	86	--------

(*b*) *Method 503.1.*—Make the test on a portion of the isolated vehicle.

(*c*) *Method 622.1.*—Apply the paint to flat tin panels with a 0.002 inch (wet film thickness) doctor blade or by other suitable means. The baking and bending constant shall be as follows:

Allow the paint to dry for 18 hours at room temperature. Bake for 5 hours at 75° plus or minus 2° C. (167° ± 4° F.). Condition the panel for a minimum of 15 minutes at 25° plus or minus 0.1° C. (77° ± 0.2° F.), and then bend at that temperature over ¼-inch mandrel. Examine for cracks in a strong light at 7 diameter magnification.

(*12*) *Dry through time.*—Prepare a panel as in paragraph 1 of Method 406.1. The film shall be considered dry through when it cannot be distorted or removed by the following test:

Place the panel in a horizontal position at a height such that when the thumb is placed on the film the arm of the operator is in a straight line from wrist to shoulder. Bear downward on the film with the full area of the ball of the thumb, exerting the maximum pressure of the arm only (not body weight), and simultaneously turn the thumb through an angle of 90 degrees in the plane of the film. Examine for loosening, detachment, wrinkling, or other evidence of distortion of the film.

(*13*) *Appearance of paint coat.*—Examine the panels prepared for brushing and spraying properties. Flow a portion of the paint on a clean glass plate. Let dry in a nearly vertical position at room temperature and examine 4 inches from the top.

(*14*) *Analysis of pigment* (*siliceous extender and mica*).—Transfer 0.5 grams of the pigment to a 400 ml. beaker. Add 15 ml. of concentrated hydrochloric acid and heat gently until the iron oxide and red lead are completely dissolved. Evaporate to dryness and bake at 105–110 degrees centigrade for 1 hour. Moisten the residue with a few drops of concentrated hydrochloric acid, dilute to 100 ml. with hot water, boil, filter, and wash with hot water. Transfer the paper and contents to a weighed porcelain crucible; ignite, cool, and weigh; and examine the residue microscopically to determine the presence of mica. Calculate the percentage and report this residue as the siliceous extender and mica.

Driver Behavior

(*Continued from page 205*)

centage of vehicles traveling below 40 miles per hour was lowest when the signs and the edge stripes were present. In general, it appears that edge stripes of the type studied n Oregon reduced vehicle speeds more than the special signs. Signs in combination with the edge stripes did not seem to influence vehicle speeds.

The primary objective for studying edge tripes and signs in Oregon was to determine what markings are effective in reducing the use of shoulders by commercial vehicles. In table 10, the average lateral positions, the percentages of vehicles encroaching on the shoulder, and the average clearances between the bodies of meeting vehicles are given for the five conditions of study. It appears that commercial vehicles used the shoulders less when only the stripes were present than when the stripes were supplemented with signs. During the daytime, more than 40 percent of the trucks encroached on the shoulder during normal operating conditions. When the stripes were present the encroachment was reduced to 13 percent and adequate clearances between the bodies of trucks meeting other vehicles were still maintained. Only 6 percent of the trucks encroached on the shoulders at night when edge stripes were used.

A 4-inch-wide solid yellow reflectorized stripe placed 13 feet from the center of a two-lane, 24-foot bituminous pavement or 1 foot on the paved shoulder was very effective in reducing the encroachment on shoulders, especially by trucks. Such edge stripes also reduced vehicle speeds about 3 miles per hour. Signs on the extreme edge of the shoulder with the legend NO TRAVELING ON PAVED SHOULDERS had only a minor effect on shoulder usage.

Trends of Factors Used in Determining the 30th Highest Hourly Traffic Volumes

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by WILLIAM P. WALKER, Highway Transport Research Engineer

Within the past two decades, highway officials have adopted the policy of designing highways to meet the traffic load of the 30th highest hourly volume of the year for which the facility was being built.

The present analysis of automatic traffic recorder data for rural highways reveals that the 30th-hour factor exhibits a tendency to decline slightly with the passing of time, rather than to remain stable as past studies have indicated. Records for 160 traffic recorder stations in continuous operation from 1946 through 1953 provided the basic data for the analysis. All classes of rural highways were represented and the coverage included 26 States.

The average factor for these stations declined at the average rate of 0.11 per year over the period 1946–53, but a wide variation in the rate of decline was found among different stations. Generally speaking, roads with volumes of more than 3,000 vehicles per day experienced a more rapid rate of decline in the factor than the roads with lesser traffic volumes. Also, 30th-hour factors of 15 or greater experienced a more rapid rate of decline than factors smaller than 15. Tabular data relating the annual change in the 30th-hour factor to both the magnitude of the factor and the average daily traffic volumes are included in this article.

IN 1940 an investigation was made of the relation between traffic volumes during peak hours and the annual average daily traffic volumes on a number of rural highways. Records from automatic traffic recorders provided the basic material for this investigation. Comparatively few of these recorders were in use at that time and none had been in continuous service for longer than 3 or 4 years. Nevertheless, a striking correlation was found between peak-hour volumes and the average daily traffic on rural highways. The authors of a report [2] on these investigations recommended that highways be designed to accommodate a volume of traffic at least as great as that which would occur during the 50th highest hour of the year, but no greater than that for the 30th highest hour. The American Association of State Highway Officials adopted the policy that highways should be designed for the 30th highest hourly volume of the year for which the highway was being built.

The work was reviewed in 1945 when better counter coverage had been established and the period of continuous operation had been extended. The results of this review [3] strengthened the recommendations of the 1940 study, and drew the additional conclusion that for any particular facility the ratio of the 30th highest hourly volume to the average annual daily volume changed very little, if at all, from year to year. This ratio, normally expressed as a percentage figure, is often referred to as the 30th-hour factor. It is a coefficient which, if known for a particular road, will yield the 30th highest hourly volume when applied to the average annual daily traffic on that road.

Purpose of Study

The present reexamination of traffic data is for the purpose of detecting any trends that may exist in the magnitude of the 30th-hour factor. If the factor for any road is indeed fixed or stable, as indications in the past have suggested, then a means is assured for estimating design-hour volumes with a degree of confidence as great as that for the estimate of the average daily traffic. If there is any tendency for the factor to become either larger or smaller with the passing of time, then the rate of change should be determined so that appropriate adjustment can be made in the design-hour volume for any future year. Unless proper adjustment of the factor is made, facilities designed for future traffic will be either overdesigned for their traffic load or they will become congested in a shorter period of time than anticipated, even though the future daily traffic is accurately predicted.

Records for 160 counters that were in continuous operation during the period 1946–53 are used in this analysis. All of these counters were located on rural highways of the 26 States shown in table 1. All classes

Table 1.—Stations included in the analysis of trends in 30th-hour factors

State	Number of stations	Numerical code assigned to stations [1]
Connecticut	18	1, 3–6, 8–11, 13, 15, 16, 18, 20–22, 25, 27
Delaware	6	1A, 2B, 3C (N and S), 4D, 5E, 7G
Georgia	4	1, 3, 4, 12
Idaho	2	3, 7
Illinois	3	2, 7, 8
Indiana	6	34A, 42A, 47A, 59A, 72A, 73A
Iowa	4	601, 604, 614, 616
Maine	5	2–4, 7, 9
Maryland	16	1–15, 18
Massachusetts	5	1, 2, 10, 12, 13
Michigan	5	600, 603, 606, 607, 617
Mississippi	4	3, 8, 10, 4A
Missouri	6	1, 2. 5, 12, 22, 26
Nebraska	5	A2, A4, A5, A8, A12
Nevada	4	101, 107, 109, 110
New Hampshire	5	1, 2, 4, 6, 15A
North Carolina	6	1–4, 22, 24
Ohio	4	25, 28, 31, 38
Oklahoma	16	1, 4–6, 8, 9, 10A, 11–15, 17–20
Pennsylvania	7	4, 5, 10, 17, 21, 22, 31
Tennessee	2	3, 11
Texas	5	1, 4, 5, 8, 17
Utah	7	301, 302, 305, 308, 312, 313, 315
Vermont	3	A12–1, C14–2, PR110
Washington	9	1–3, 6–8, 12, 14, 15
West Virginia	3	1, 3, 4
Total	160	

[1] Individuals or agencies having a bona fide interest may obtain detailed traffic data for any station or group of stations identified in this column by addressing the U. S. Bureau of Public Roads, Division of Highway Transport Research, Washington 25, D. C.

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[1] This article was presented at the 36th Annual Meeting of the Highway Research Board, Washington, D. C., January 1957.

[2] *Applications of automatic traffic recorder data in highway planning*, by L. E. Peabody and O. K. Normann. PUBLIC ROADS, vol. 21, No. 11, January 1941.

[3] *Highway Capacity Manual.* Published by the Bureau of Public Roads, 1950, pp. 129–147.

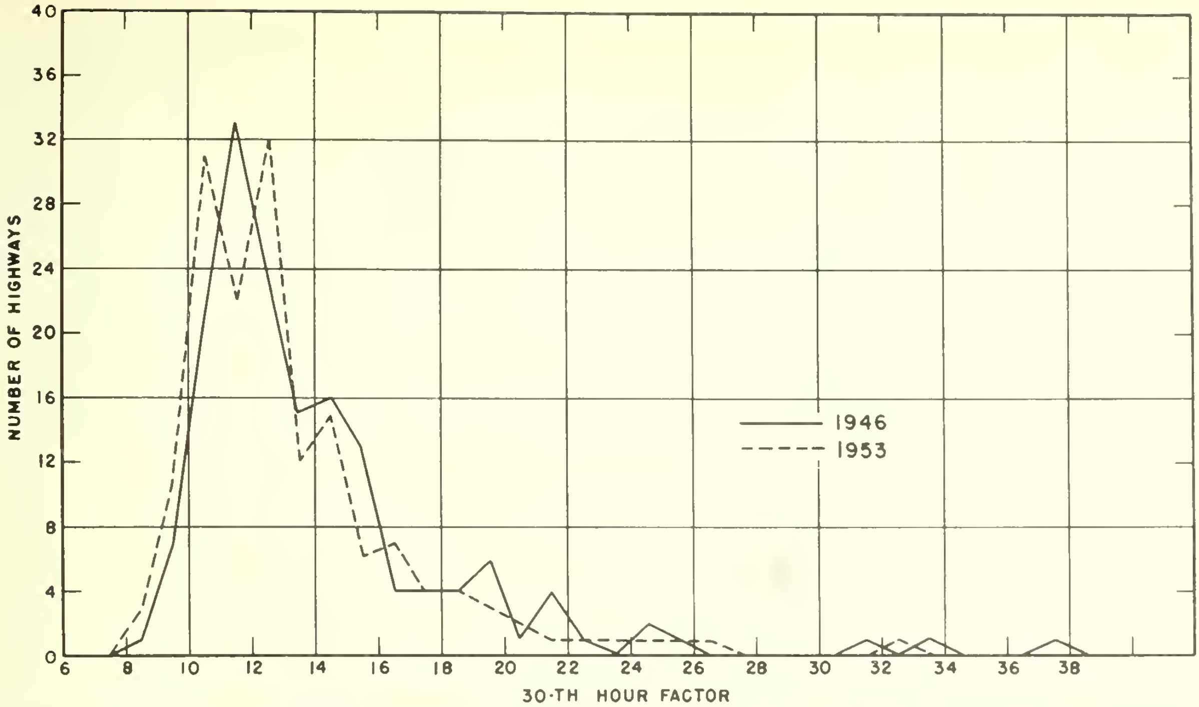

Figure 1.—Distribution of 30th-hour factors for 160 rural highways of all classes in 26 States for the years 1946 and 1953.

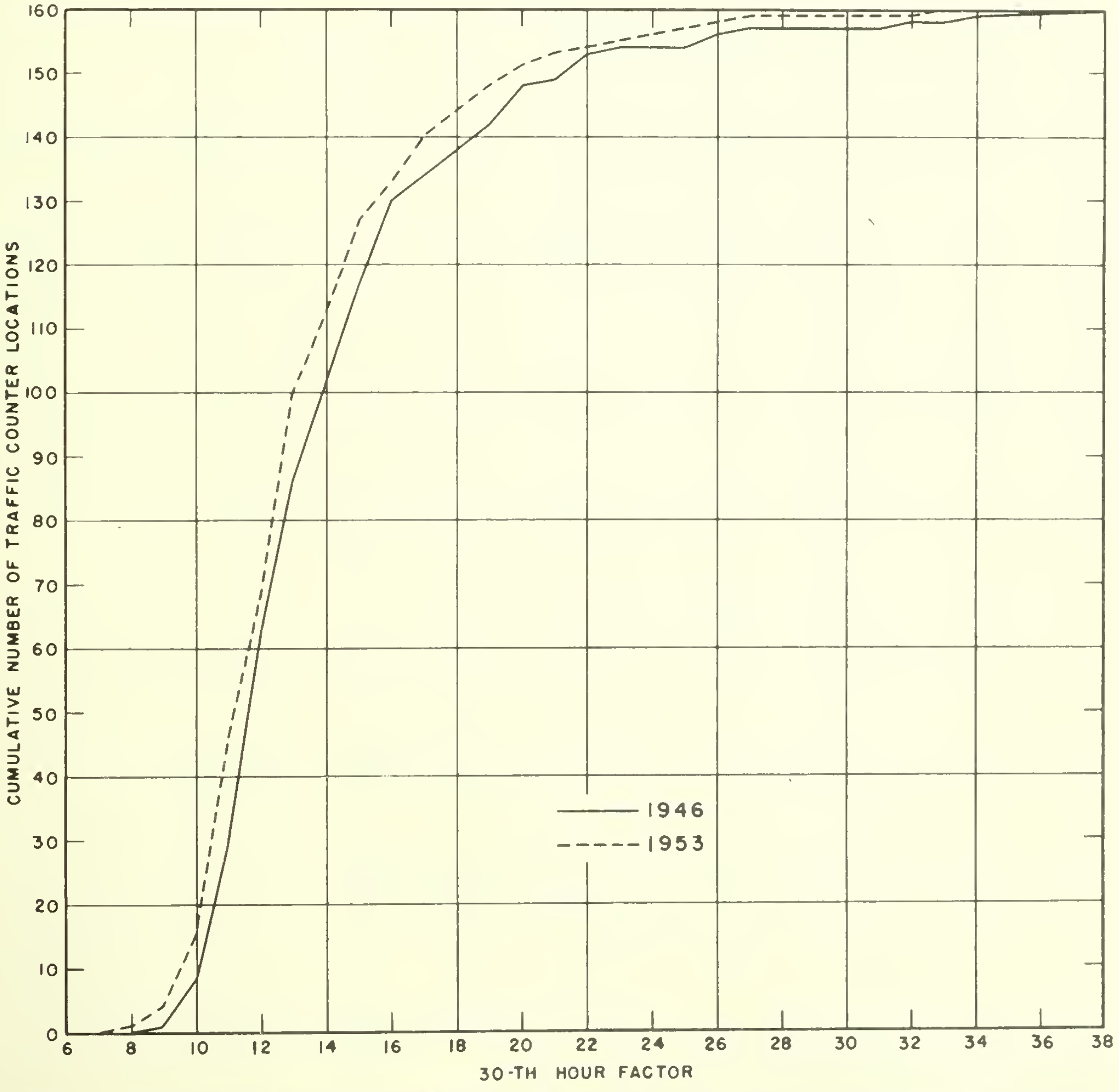

Figure 2.—Cumulative number of traffic counter locations having 30th-hour factors of various values for the years 1946 and 1953.

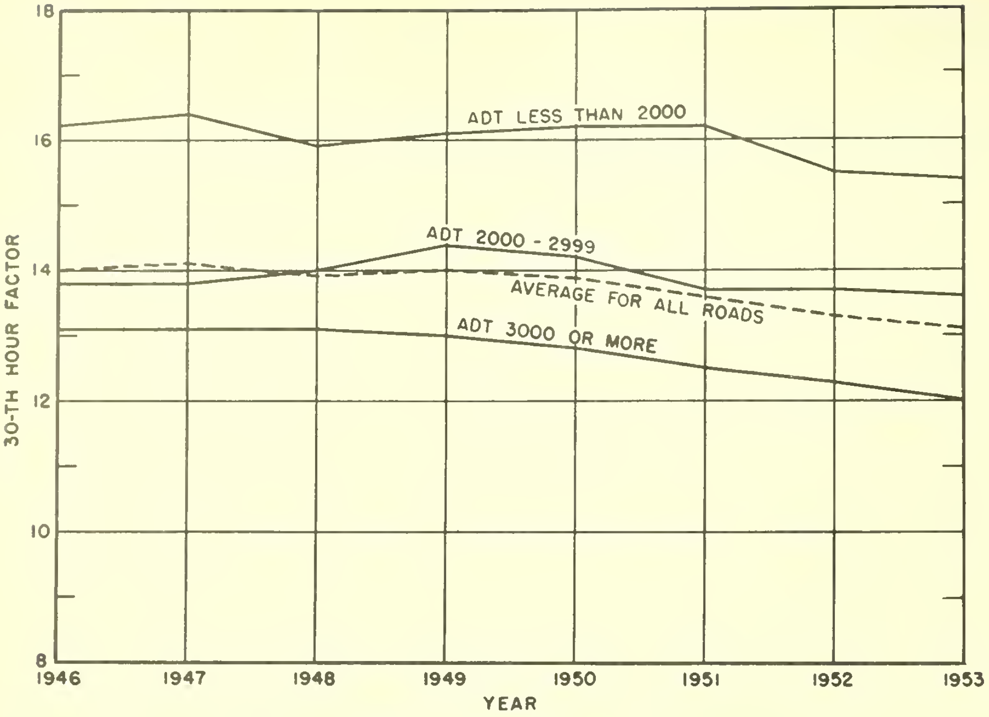

Figure 3.—Trends in the magnitude of 30th-hour factors for three traffic volume groups for the years 1946–53.

of rural roads were represented and the range in traffic volumes was from a few hundred vehicles per day on the most lightly traveled roads to more than 25,000 vehicles per day on the more heavily traveled multilane roads.

Wide Range in Factors at Different Locations

For the year 1946, the 30th-hour factors for the 160 locations ranged in magnitude from less than 9.0 to above 30.0. The frequency of occurrence of factors of various magnitudes is shown in figure 1 for 2 years, 1946 and 1953. The similarity between the distributions of factors for the 2 years is very striking. The distribution curve for 1953 is located slightly to the left of the one for 1946, and this is an indication that the factors for the more recent year are somewhat smaller than they were in 1946. The fact that a reduction in the factor has occurred during the 1946–53 period is more readily apparent in figure 2. This figure shows the cumulative number of traffic counter locations for which the 30th-hour factors are equal to or less than the values specified on the horizontal scale. Data for the 2 years, 1946 and 1953, are again represented.

A condition not shown in figure 2 is that factors for some stations changed by a far greater amount than others. Forty-six of the 160 stations actually experienced an increase in the factor during the years 1946–53. For the remaining 114 stations, or 71 percent of the total, the factor showed a decline. The average factor changed from 14.07 in 1946 to 13.25 in 1953, a decrease of 0.82.

Greatest Change in Factors Occurs on High-Volume Roads

On the average, the roads carrying relatively low volumes (below 2,000 ADT) had the highest factors, and those carrying the heaviest volumes had the lowest factors. This is illustrated in figure 3 where the stations have been divided into three volume groups: those below 2,000 ADT, those between 2,000 and 3,000, and those over 3,000 ADT. These were the volumes in 1951.

Roads within the volume range of 2,000 to 3,000 vehicles per day had the least change in the factor between 1946 and 1953. In 1946 the factor for this group was 13.8 as compared with 13.6 in 1953. The trend throughout the period was far from uniform, and it is questionable whether the 18 stations in this group constitute a sufficient sample to justify firm conclusions. In a subsequent paragraph where the effect of the magnitude of the factor is discussed, it is shown that the sample is not only small but is biased in favor of stations having low factors.

For roads having a daily volume of 2,000 vehicles or less, the average factor declined from 16.2 in 1946 to 15.4 in 1953, a change of −0.8. There were 43 stations in this group. Roads having an average daily volume of over 3,000 vehicles showed the greatest change of all the groups. For these roads the average factor was 13.1 in 1946 and 12.0 in 1953. The difference between the factors for these years is 1.1. A total of 99 stations comprise this group and the trend over the years appears to be very consistent.

The curves in figure 3 suggest that the change in the factor with the passage of time is almost nil for roads carrying moderately heavy traffic; that is, between 2,000 and 3,000 vehicles per day. As a class, roads carrying fewer than 2,000 vehicles daily have shown a decline in the 30th-hour factor of about 0.11 per year. For roads carrying more than 3,000 vehicles daily, the factor has shown the rather marked rate of decline of 0.16 per year.

Roads Having Low Factors Show Least Change

If the apparent finding just stated were applied indiscriminately, the 30th-hour factors for our most heavily traveled roads would rapidly approach the absolute minimum, which is theoretically 4.15, assuming the traffic volume is constant during all hours of the year. None of the stations in the sample had a factor anywhere near this small—the lowest one being 8.2 in 1946 for a heavily traveled multilane facility in a densely populated section of the country. Only six stations had factors in 1946 of less than 10.0. For this group of six stations the change in the average factor during the period 1946–53 has been almost imperceptible. This is illustrated by the lower curve in figure 4, which shows the trend in the factor for five ranges in the values for the 30th-hour factor.

It is only for the groups of factors having

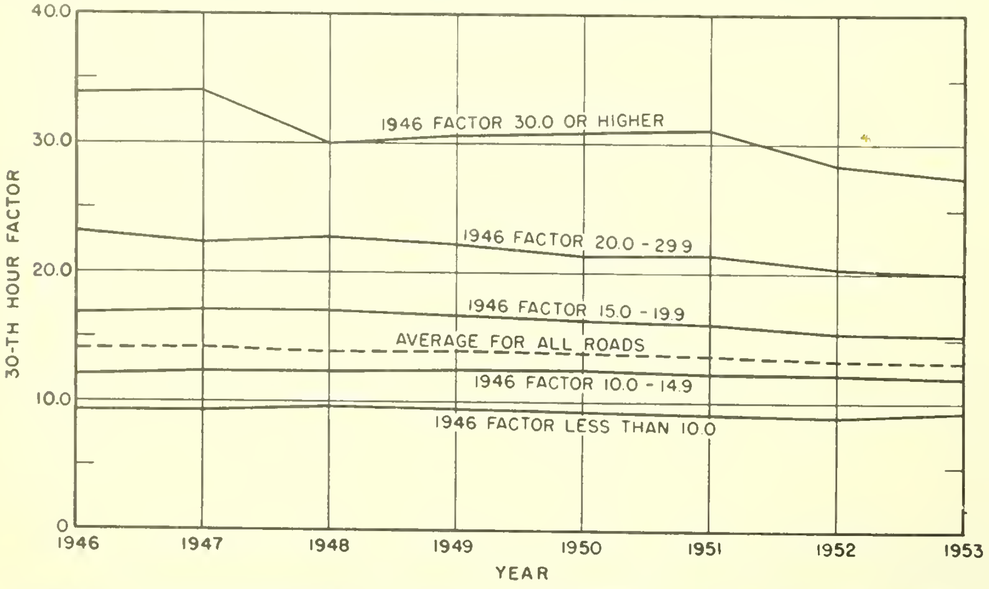

Figure 4.—Trends in the 30th-hour factor for roads having factors of various magnitudes for the years 1946–53.

values of 15.0 or more that any appreciable change in the average factor with passage of time is to be noted. The stations having the highest factors are those showing the greatest change. Average factors for stations having a 1946 factor of 30 or more have declined from 33.7 in 1946 to 27.4 in 1953, a reduction of 6.3. For stations having 1946 factors between 20 and 30, the change in the average factor during the period amounts to 3.0, while the corresponding change for stations having 1946 factors within the range of 15.0–19.9 is 1.8. For factors of less than 15.0, which comprise 72 percent of the total number of stations studied, the change in the factor with passage of time is negligible. This further accounts for the apparent stability of the average factor for roads having volumes between 2,000 and 3,000 ADT, as shown in figure 3, because 14 of the 18 stations within this volume group had factors smaller than 15.0.

The average of the factors for the six stations having 30th-hour factors of less than 10.0 suggests that as a practical matter the irreducible minimum value must be in the neighborhood of 9.5. There are, of course, exceptions to this rule but they are few and limited perhaps to facilities that are so heavily overladen that travel habits are influenced excessively by a high degree of congestion.

Group Factors Decline Although Some Stations Show Increase

The foregoing discussion has shown that if the stations are classified into groups according to either traffic volume or magnitude of factor, the average of the factors for any group shows at least a slight tendency toward decreasing with passage of time. It has also been stated that when the stations are considered individually, almost 30 percent of them experienced an increase in the 30th-hour factor. When the stations are grouped according to magnitude of factor, some of the groups will include stations for which there was an increase in the factor, but it is evident that the effect of these stations is more than offset by the stations within the group that experienced a decrease in the factor.

The groups of stations that showed the least change in the factor—the groups having factors of less than 15—were those that included most of the stations for which there was an increase in the factor. Figure 4 shows that the groups of stations having factors of less than 15 experienced very little change in the average factor. Of the 115 stations for which the factor in 1946 was less than 15.0, 41 stations experienced an increase and 74 experienced a decrease in the factor. Of the 45 stations having factors of 15 or more, only 5 experienced an increase in the factor. Thus, if the factor for any station is less than 15, there is better than one chance in three that the factor will increase rather than decrease with the passage of time. Also, if the factor is greater than 15, there is very little likelihood that it will increase at all.

A somewhat different and more refined interpretation can now be made of figure 3 than was permissible prior to examination of figure 4. It is now apparent that the magnitude of the factor and the traffic volume act in combination to influence the trend in the factor with passage of time. It has been established that factors for roads carrying volumes in excess of 3,000 ADT decline at a more rapid rate than those with lesser volumes and, additionally, that factors of 15 or more will decline at a more rapid rate than will smaller factors. The magnitude of the factor seems to exert a greater influence on the rate of change than does traffic volume. Roads with volumes in excess of 3,000 ADT in combination with factors of 15.0 or more experience the highest rate of decline in the factor. The lowest rate of decline in the factor is experienced by roads having factors of less than 15.0, regardless of the traffic volume. Perhaps there are numerous variables other than traffic volume and magnitude of factor which influence the trend, but the amount of information available for this analysis imposes rather stringent limitations on their detection and evaluation.

With the passing of years the number of stations having high factors within a given sample will diminish and the average rate of decline will be reduced. When the factor for a particular facility reaches a value somewhere between 9.5 and 15.0, depending upon such things as the character of traffic and geographic location, the decline in the factor will be arrested and very little change may be expected thereafter. If the conditions surrounding the minimum values lying within the range of 9.5 to 15.0 could be ascertained, the results of this analysis would be greatly enhanced, but this does not appear to be possible from the data at hand.

Summary

The findings thus far discussed are as follows:

1. The trend in the 30th-hour factor for 160 stations in continuous operation over the 1946–53 period has been a decline of 0.11 per year, on the average, but the rate of decline varied widely among different roads.
2. Roads having volumes greater than 3,000 ADT in combination with factors of 15 or above are those which, as a class, experience the most rapid rate of decline in the factor.
3. The lowest average rate of decline (almost zero) is experienced by roads having 30th-hour factors of less than 15.0, regardless of traffic volume.
4. The fact that there is little change in the average factors for the groups of roads having factors of less than 15.0 is due in part to the phenomenon that within these categories there is almost a one-in-three chance that any given station will experience an increase rather than a decrease in the factor with passage of time.
5. The irreducible minimum value for the 30th-hour factor may be taken as about 9.5 for rural highways. In some geographical areas and under certain conditions yet to be defined, the minimum value of the factor may be as high as 15.0, but a factor lower than 9.5 may be accepted as a definite indication that travel desires are being suppressed.

Application of Findings

As in the case of many investigations of this type the best that can be hoped for is the development of broad indications which, if applied, will provide results that are more nearly correct than would have been possible in the absence of the study. To be of maximum value to the user, the findings of the investigation must be accurate and in considerable detail. When measured against this standard, the study reported here is somewhat deficient because it does not answer fully the question as to why traffic patterns for some roads behave differently from others in the course of time. If applied wisely, however, such facts as have been brought to light should permit a far more accurate estimate of future 30th-hour factors than would otherwise be possible.

As an aid in applying the results of the study, a table of suggested annual changes in the 30th-hour factor for various combinations of factors and traffic volumes has been prepared. These suggested annual changes, as shown in table 2, approximate the average changes found in the statistical analysis. For traffic volumes other than those shown in the column headings of table 2, interpolation between columns is recommended. The suggested method is given in the footnote to table 2 and in the two examples that follow. The volumes of 1,500, 2,500, and 3,500 ADT, shown in the column headings of the table, were selected to aid the user of the data in

Table 2.—Annual change in 30th-hour factor for various combinations of factors and traffic volumes

30th-hour factor	Annual change in 30th-hour factor [1] for ADT of—		
	1,500	2,500	3,500
Pct.	*Pct.*	*Pct.*	*Pct.*
Below 10.0	0.00	0.00	−0.08
10.0–10.9	.00	.00	−.10
11.0–11.9	.00	.00	−.12
12.0–12.9	.00	.00	−.15
13.0–13.9	.00	−.01	−.18
14.0–14.9	−.01	−.02	−.22
15.0–15.9	−.02	−.04	−.27
16.0–16.9	−.03	−.07	−.31
17.0–17.9	−.05	−.10	−.36
18.0–18.9	−.08	−.13	−.41
19.0–19.9	−.10	−.17	−.48
20.0–20.9	−.14	−.20	−.53
21.0–21.9	−.18	−.24	−.59
22.0–22.9	−.21	−.29	−.65
23.0–23.9	−.25	−.34	−.71
24.0–24.9	−.30	−.39	−.79
25.0–25.9	−.35	−.44	−.83
26.0–26.9	−.40	−.50	−.90
27.0–27.9	−.46	−.55	------
28.0–28.9	−.52	−.61	------
29.0–29.9	−.58	−.67	------
30.0–30.9	−.63	−.74	------
31.0–31.9	−.70	−.81	------
32.0–32.9	−.78	−.90	------
33.0–33.9	−.83	------	------
34.0–34.9	−.90	------	------

[1] For volumes of less than 1,500 ADT, use values shown in column 2; for volumes within the range of 1,500 and 2,500 ADT, interpolate between values shown in columns 2 and 3; for volumes within the range of 2,500 and 3,500 ADT, interpolate between values shown in columns 3 and 4; for volumes of 3,500 ADT and over, use values shown in column 4.

interpolating; actually, the average daily volumes for the groups of stations exhibiting annual changes in the 30th-hour factor were 1,200, 2,500, and 6,500, respectively.

Table 3.—Examples of the application of annual changes in the 30th-hour factor for various combinations of factors and traffic volumes

Year	Example 1			Example 2		
	Average daily traffic	30th-hour factor	Annual change	Average daily traffic	30th-hour factor	Annual change
1956	5, 300	14. 30	−0. 22	1, 400	18. 50	−0. 08
1957	5, 560	14. 08	−. 22	1, 550	18. 42	−. 08
1958	5, 850	13. 86	−. 18	1, 700	18. 34	−. 09
1959	6, 150	13. 68	−. 18	1, 875	18. 25	−. 10
1960	6, 460	13. 50	−. 18	2, 050	18. 15	−. 11
1961	6, 790	13. 32	−. 18	2, 250	18. 04	−. 12
1962	7, 130	13. 14	−. 18	2, 500	17. 92	−. 10
1963	7, 500	12. 96	−. 15	2, 750	17. 82	−. 17
1964	7, 880	12. 81	−. 15	3, 000	17. 65	−. 23
1965	8, 280	12. 66	−. 15	3, 300	17. 42	−. 31
1966	8, 700	12. 51	−. 15	3, 600	17. 11	−. 36
1967	9, 140	12. 36	−. 15	3, 950	16. 75	−. 31
1968	9, 600	12. 21	−. 15	4, 400	16. 44	−. 31
1969	10, 100	12. 06	−. 15	4, 800	16. 13	−. 31
1970	10, 600	11. 91	--------	5, 300	15. 82	--------

Example 1

The present (1956) volume on a rural road is 5,300 ADT. The 30th highest hourly volume in 1956 was 778 vehicles per hour, yielding a 30th-hour factor of 14.30. It is estimated that the ADT in 1970 will be 10,600, or double the present volume. Estimated volumes for the intervening years between 1956 and 1970 are as shown in column 2 of table 3 What will be the 30th-hour factor for 1970?

Solution.—Since the volume for all years is above 3,500, column 4 of table 2 applies to this example. No interpolation is required. The annual change for 1956 is found in column 4 of table 2. The change of −0.22 is applied to the 1956 factor of 14.30 to yield a 1957 factor of 14.08. Thirtieth-hour factors for the succeeding years are obtained in a similar manner. The rate of change diminishes as the factor becomes smaller.

Example 2

The present (1956) volume on a rural road is 1,400 ADT. The 30th-hour volume in 1956 was 259 vehicles per hour, yielding a 30th-hour factor of 18.50. The estimated rate of traffic growth is about 10 percent per year in the following 14 years, as indicated in column 5 of table 3. What will be the 30th-hour factor in 1970?

Solution.—Since the ADT changes from an initial value of less than 1,500 to a final value of over 3,500, columns 2, 3, and 4 of table 2 are used in this example. The factor for 1957 is obtained by subtracting the annual change for 1956 (0.08, column 2, table 2) from the 30th-hour factor for 1956 (18.50). The procedure is repeated for each succeeding year to 1970, with the resulting 30th-hour factor for 1970 being 15.82. During the period 1957–61, the ADT is between 1,500 and 2,500 and the annual change is obtained by interpolating between −0.08 in column 2 of table 2 and −0.13 in column 3. During the period 1963–65, the ADT is between 2,500 and 3,500 and the annual change is determined by interpolating between −0.10 in column 3 and −0.36 in column 4. For the period 1966–69, the volume is greater than 3,500 and no interpolation is required. For these years the annual change is taken directly from column 4 of table 2.

AE–55 Indicator

(*Continued from page 207*)

indicator gave results averaging about 1 percentage point too high. For air contents of over 6.0 percent the AE–55 indicator gave values averaging 1 percentage point too low. These values indicate that it might be feasible to prepare a correction curve for the AE–55 indicator readings. Figure 4 indicates the amount by which the reading for the AE–55 indicator should be corrected to agree with values for the pressure meter. In this figure the circles indicate the results for each mix, whereas the crosses are the average values given in table 2.

Field Testing of AE–55 Indicator

A limited number of tests were made in the field to determine the accuracy and usability of the apparatus on a paving job. Tests were made on concrete containing 4 percent air as determined by a pressure air meter. The values obtained with the AE–55 indicator were between 3.5 and 4.5 percent with no correction for the mortar content. The corrected values would be from 3.2 to 4.1 percent. If these values were further corrected, as shown by the curve in figure 4, the values obtained would be 3.9 to 4.3 percent which agree closely with the pressure air meter determination. The determinations were made by four different engineers, three of whom had not seen the apparatus before. The device was very favorably received because of its small size and the rapidity with which the test could be made in the field.

Summary

The AE–55 indicator is found to be an apparatus of considerable merit for use in the determination of the approximate air content of concrete in the field, provided the amount of mortar in the concrete is known. The test can be completed in less than 3 minutes and the apparatus can be carried in one's pocket. Attention is called, however, to the small amount of mortar used in a test. To insure the most reliable results, at least three tests should be made for each determination of air content.

The AE–55 indicator method is considered not suitable as a replacement of the pressure or gravimetric methods for the air content of concrete, but is useful as a supplementary test. It does appear to be of most value for use in determining the uniformity of the air content from batch to batch of concrete when no change in the materials or proportions occurs. It may also be used as a rapid check to determine whether the air content is within the specification limits. In no case, however, should the AE–55 indicator method be considered suitable for replacing any of the standard methods mentioned.

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF JUNE 30, 1957

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			CONTRACTS ADVERTISED, CONSTRUCTION NOT STARTED			PROJECTS UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$18,435	$36,282	$28,403	255.7	$10,131	$5,844	108.7	$77,858	$48,140	955.4	$124,271	$82,387	1,319.8
Arizona	1,396	24,178	21,594	180.0	9,958	8,502	76.3	20,439	16,834	113.3	54,575	46,930	369.6
Arkansas	17,866	40,278	29,260	533.4	14,149	8,514	125.2	33,635	19,141	464.2	88,062	56,915	1,122.8
California	2,668	34,295	14,143	250.5	42,208	31,795	52.4	404,617	203,334	289.9	481,120	249,272	592.8
Colorado	19,706	16,679	12,281	136.6	13,390	9,649	73.3	40,268	27,360	278.4	70,337	49,290	488.3
Connecticut	33,091	2,771	1,474	4.0	26,368	15,940	18.3	13,358	8,869	26.5	42,497	26,283	48.8
Delaware	16,652	2,350	1,203	14.1	3,921	2,198	28.3	14,331	7,955	51.9	20,602	11,356	94.3
Florida	24,745	24,641	19,407	178.2	18,584	12,752	38.1	38,045	21,696	235.7	81,270	53,855	452.0
Georgia	41,552	54,712	34,015	681.6	8,186	4,399	50.1	89,329	50,583	834.1	152,227	88,997	1,565.8
Idaho	20,007	15,691	12,419	117.3	4,986	4,218	32.8	16,873	11,360	199.8	37,550	27,997	349.9
Illinois	26,378	75,227	51,418	569.9	59,991	47,475	75.8	150,410	102,386	883.1	285,628	201,279	1,528.8
Indiana	84,007	24,501	13,295	134.0	15,903	8,344	95.9	44,270	28,010	168.9	84,674	49,649	398.8
Iowa	3,582	52,024	40,277	472.9	11,830	7,142	183.2	67,095	43,097	1,451.7	130,949	90,516	2,107.8
Kansas	5,733	48,570	39,828	897.4	15,157	9,448	287.8	39,188	23,103	1,185.0	102,915	72,379	2,370.2
Kentucky	46,586	10,002	6,381	85.6	4,563	2,944	11.9	50,670	33,647	317.0	65,235	42,972	414.5
Louisiana	26,115	41,263	26,892	173.1	13,910	8,828	72.9	50,176	26,691	323.2	105,349	62,411	569.2
Maine	23,818	8,999	4,842	83.2	2,929	1,560	16.4	18,966	10,085	118.6	30,894	16,487	218.2
Maryland	3,435	24,282	12,482	82.4	16,384	11,638	18.1	62,351	41,303	203.8	103,017	65,423	304.3
Massachusetts	26,648	40,602	25,550	35.0	39,794	26,830	22.6	64,655	35,208	55.1	145,051	87,588	112.7
Michigan	17,818	71,514	55,535	650.2	35,676	19,335	80.7	112,433	75,942	473.7	219,623	150,812	1,204.6
Minnesota	15,686	16,458	13,410	322.1	15,492	11,341	87.8	86,795	59,391	1,424.9	118,745	84,142	1,834.8
Mississippi	6,396	37,607	26,728	685.7	24,078	19,716	121.9	40,745	23,944	814.6	102,430	70,388	1,622.2
Missouri	27,130	30,956	16,173	1,161.7	17,267	13,683	21.6	118,475	72,646	1,374.1	166,698	102,502	2,557.4
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Oregon	18,003	8,899	7,207	58.0	4,266	3,516	19.3	44,289	31,922	281.8	57,454	42,645	359.1
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Texas	86,382	19,329	13,125	379.7	52,150	38,066	169.5	177,528	113,182	1,710.8	249,007	164,373	2,260.0
Utah	15,132	17,274	14,034	143.3	2,673	1,991	13.8	15,709	12,489	191.2	35,656	28,514	348.3
Vermont	10,627	4,505	3,057	24.5	235	115	0.4	20,123	13,522	67.9	24,863	16,694	92.8
Virginia	25,727	47,880	37,132	174.0	16,784	9,478	136.8	43,076	24,228	306.3	107,740	70,838	617.1
Washington	24,513	22,351	16,044	210.9	4,796	3,846	48.4	50,577	33,320	303.0	77,724	53,210	562.3
West Virginia	18,527	43,774	31,859	66.0	4,018	2,049	10.9	31,375	16,177	96.1	79,167	50,085	173.0
Wisconsin	52,444	17,097	9,417	356.0	8,131	4,213	60.5	64,833	38,329	467.1	90,061	51,959	883.6
Wyoming	2,089	9,911	8,170	58.3	7,514	5,969	55.1	39,359	30,582	401.7	56,784	44,721	515.1
Hawaii	5,116	4,987	2,493	14.7	10,840	7,925	4.6	4,340	2,076	8.5	9,327	4,569	23.2
District of Columbia	14,791	6,125	4,886	2.9				11,447	8,624	0.6	28,412	21,435	8.1
Puerto Rico	11,536	4,138	1,558	12.8				18,171	8,614	56.1	22,309	10,172	68.9
Alaska	479	11,099	10,025	104.5	2,813	2,577	354.5	2,136	1,953	108.8	16,048	14,555	567.8
TOTAL	1,182,622	1,486,213	1,035,374	13,994.4	842,128	573,096	3,825.6	3,521,853	2,154,649	24,141.2	5,850,194	3,763,119	41,961.2

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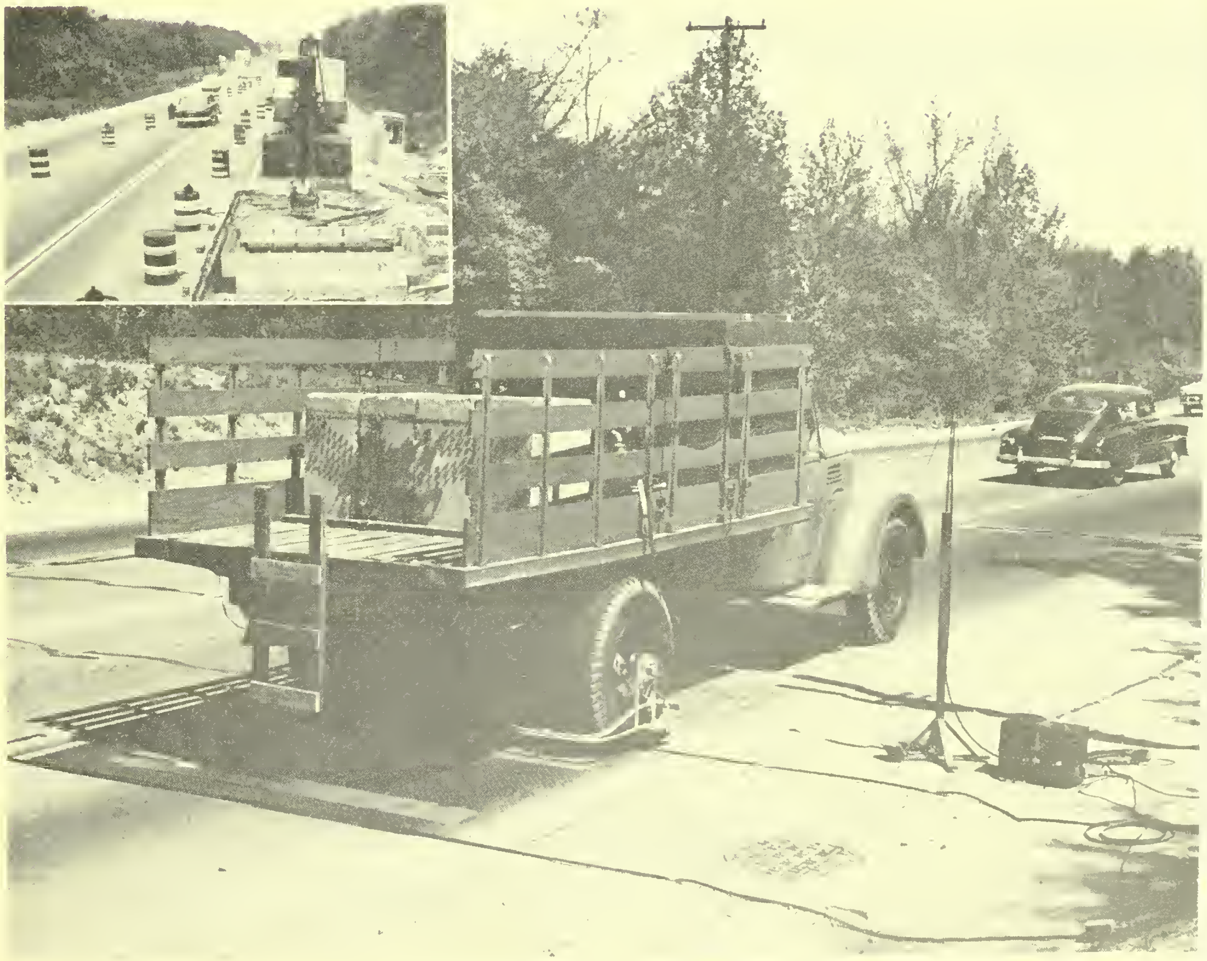

In this issue: A comparison of methods for measuring loads transferred through vehicle tires.
(Inset: Installation of electronic scale)

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The printing of this publication has been approved by the Director of the Bureau of the Budget, Mar. 17, 1955.

Contents of this publication may be reprinted. Mention of source is requested.

A Comparison of Methods Used for Measuring Variations in Loads Transferred Through Vehicle Tires to the Road Surface

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by RICHARD C. HOPKINS, Electrical Engineer, and HOWARD H. BOSWELL, Highway Research Engineer

An earlier study involving the weighing of vehicles moving over the highways by means of an electronic scale gave promise of an efficient enforcement tool of the States in detecting vehicles carrying loads in excess of the legal limit. It was found at that time that the electronic scale was quite reliable in measuring gross weights of vehicles, but in determining the weights of individual axles the error exceeded 5 percent for about 35 percent of the axles weighed.

The axle weights recorded by the electronic scale while the vehicles were in motion were sometimes greater, smaller, or equal to the static weights. Such a random pattern suggested that trucks undulate on their springs while in motion. As a result the axle weights recorded by the electronic scale varied with the amplitude and frequency of the oscillations and depended upon what part of the cycle was measured on the electronic scale platform. The amplitude of the oscillations increased with the roughness of the roadway and the highest frequencies occurred on the smoother surfaces.

In this article, three different methods for obtaining a continuous record of the magnitudes and variations in the weights transferred through the tires to the road surface are discussed in conjunction with weights recorded by the electronic scale. Comparisons are made of the changes in the axle-housing strain, changes in tire sidewall deflection, and changes in the air pressure within the tire as a result of variations in wheel loads. Of the three methods studied, the air pressure method proved to be the most practical. Dual-tire equipment introduced an additional factor of a changing lever arm in measuring axle-housing strain, and the installation of sidewall detectors between the tires of a dual assembly for measuring deflection was exceedingly difficult.

THE load transmitted to the road surface through the tires of a vehicle in motion s not constant but varies in frequency and magnitude with several factors such as the speed of the vehicle, the smoothness of the road surface, tire pressure, and the vehicle's suspension system. An adequate means for obtaining a continuous record of the frequency and magnitude of these load variations has not been available.

The development of the electronic scale has not only made it possible to weigh vehicles in motion but also has made available a means for measuring the load transmitted to the road surface over a limited distance, thus providing a means for checking the accuracy of other methods that could be employed on a vehicle to obtain a continuous record.

The results of studies made of the electronic scale [2] near Woodbridge, Va., showed that while most vehicles in motion registered close to their static weight and there was little error in the gross tonnage, the error was in excess of 5 percent for about 35 percent of the individual axles weighed. The dynamic weight was just as likely to be either heavier or lighter than the static weight.

This suggests that trucks are undulating on their springs while moving over the highways. The axle weight as recorded by the electronic scale would depend on the amplitude of this oscillation and what part of the oscillation cycle was measured by the scale. The amplitude can be controlled somewhat by a smooth pavement surface, but the measured part of the cycle cannot be fixed and the scale will record random parts of the cycle. This would account for some axles being recorded as heavier, some lighter, and some equal to their static weight.

Purpose of Study

This report is the result of a study to determine, if possible, the best means for measuring the magnitude and frequency of these variations in weight. Some preliminary work had been done in connection with the WASHO Road Test.[3] Information was also obtained to determine whether the electronic scale records the changes accurately, or whether there is an error in the basic design of the electronic scale.

The electronic scale used for these investigations was recently constructed in the northbound lane of U. S. Route 1 about 1,000 feet south of the Virginia State Official Weighing Station near Woodbridge. The scale was constructed as a cooperative project of the Virginia Department of Highways, a manufacturer of electronic scales, and the Bureau of Public Roads in an effort to improve the accuracy of electronic scales and to determine the feasibility of their use in connection with the enforcement of legal weight limits.

The scale has a 7-foot platform in the direction of traffic and extends across the right-hand lane of the two northbound lanes. It embodies all the improvements that were determined as a result of research conducted on the first electronic scale constructed for weighing vehicles in motion on the Shirley Memorial Highway (Virginia) in April 1951.

In conjunction with the electronic scale which records wheel loads at one place on a highway, three different methods for obtaining a continuous record of the magnitudes and variations in the weight transferred by the tires to the road surface were developed and tested simultaneously. These were as follows: changes in the axle-housing strain near the spring mounting; the change of bulge or spread of the tire sidewalls directly above the center of contact with the pavement surface; and the changes in air pressure within the tire.

The vehicle, shown in figure 1, on which the instruments were mounted for the three types of measurements is a 2-axle single-unit truck with 9.00 x 20 tires. The inside tire on each dual wheel was removed to permit the installation of the tire sidewall detectors. Four 1,000-pound concrete blocks were placed on

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[1] This article was presented at the 36th Annual Meeting of the Highway Research Board, Washington, D. C., January 1957.

[2] *Weighing vehicles in motion*, by O. K. Normann and R. C. Hopkins. PUBLIC ROADS, vol. 27, No. 1, April 1952.

[3] *The WASHO Road Test, Part 2: Test Data, Analysis, and Findings.* Highway Research Board, Washington, D, C., 1955, Special Report 22, Section 5e, pp. 170-174.

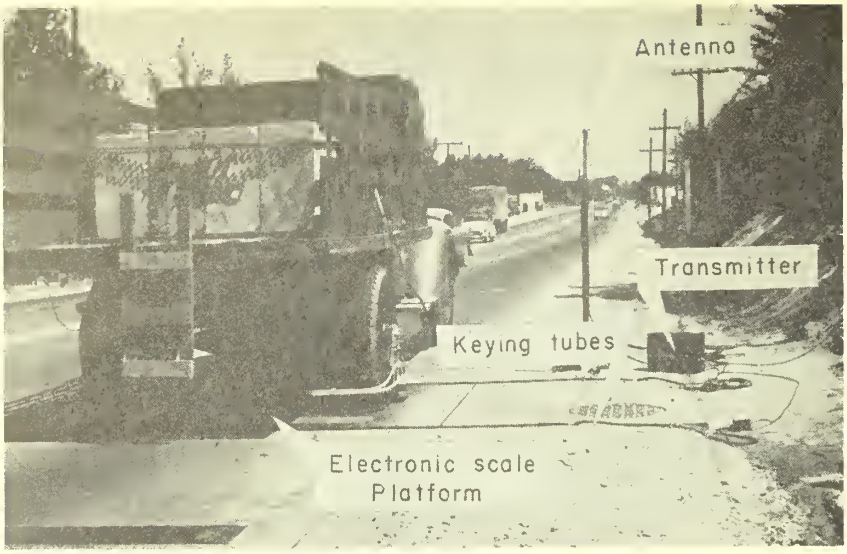

Figure 1.—Single-unit truck equipped with apparatus used in measuring wheel loads.

the truck bed to give a 9,000-pound rear-axle load. This produced a tire load of 4,500 pounds which is equal to that allowed by the many States having an 18,000-pound single-axle load limit. The right rear wheel only was instrumented.

Conclusions

1. The most practical of the three methods for measuring the weight variations involving all wheels of a vehicle is the tire pressure method. The changing lever arm for dual tires, when the axle-housing strain is measured, and the difficulty of mounting rollers between dual wheels to measure tire sidewall deflection make these two methods less desirable. It is possible by incorporating the refinements outlined in the subsequent discussion of measuring the changes in tire air pressure to continuously record, with acceptable accuracy, the force exerted on the pavement by each tire of a vehicle.

2. Pavement deflections can now be related to the actual load applied by a vehicle in motion.

3. It is also possible to compare the load transfer characteristics of the many different suspension systems now in common use on the undercarriage of trucks.

4. Differences in weight of more than 5 percent between the static axle weight and the dynamic weight as shown by the electronic scale with a stable platform are probably due to the undulating load of a vehicle in motion rather than to errors of the scale.

Axle-Housing Strain

Four strain gages were used as the arms of a resistance bridge circuit to record the changes in axle-housing strain due to variations in the load on the wheel. The gages were cemented to the rear axle housing just inside the spring mount, two gages on top and two on the botton as shown in figure 2. The axle housing is a continuous cantilever beam which transfers the load on the springs to the wheel resting on the pavement. Any change in the wheel load causes a change of strain in the axle housing.

The calibration of the axle strain gages was easily done. The wheel of the vehicle was placed on a scale platform, a loadometer in this case, and the load changed on the bed of the truck. For each change in load the recording instruments registered a corresponding change in strain, which then gave a calibration curve.

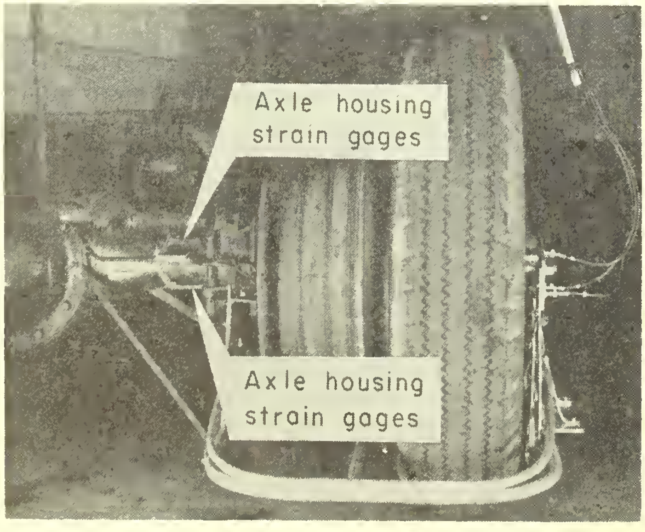

Figure 2.—A view of the mounting of the 4 strain gages on the rear axle housing.

This is a simple means of recording the change in wheel load, and it worked very well in this study. There are, however, factors which limit its usefulness. As the pavement surface becomes rough and the truck speed increases, the axle-housing strains are influenced not only by the load resting on the springs, the sprung load, but also by the weight or inertia of the truck undercarriage, the unsprung load. On a smooth surface and at slow speeds the unsprung load would have little or no effect since it would have practically no vertical motion or acceleration.

Another complicating factor is the addition of the second wheel of a dual assembly. The addition of this second wheel would change the lever arm of the cantilever axle-housing beam, depending upon which of the two tires was carrying the greater proportion of the momentary load. Therefore, the calibration would continually change due to the changing lever arm. Again, on a smooth surface this would have little or no effect because each tire would carry the same proportion of the load, provided the inflation pressure of each was the same.

Bulge or Spread of Tire Sidewalls

The second method of determining wheel-load changes was suggested by Endres and Bombard.[4] As the load increases on a pneumatic tire, the lower part of the tire casing bulges or spreads due to its contact with the pavement surface. This tire sidewall deflection can be calibrated in a manner similar to the axle-housing strain, with the exception that the wheel must be rotated between each change in load to allow the sidewall to attain its normal deflection for the load as it would with the vehicle in motion.

Equipment Used to Measure Deflection

Figure 3 shows the instrument used to measure the tire sidewall deflection. This assembly was built in the Bureau of Public Roads laboratory and consisted of two 1/16 x ¾-inch pieces of spring steel 9½ inches long riveted to a ¼ x 3 x 3-inch aluminum plate. The two pieces of spring steel were spaced ¾ inch apart to allow for the mounting of a 200 SFF ball bearing which acted as a roller running against the tire sidewall. The ball bearing was mounted between the two pieces of spring steel which were used to measure the spread of the tire sidewall by the amount that the roller caused them to bend. Thus, when the plate was held rigid and the roller was in contact with the tire sidewall, any deflection

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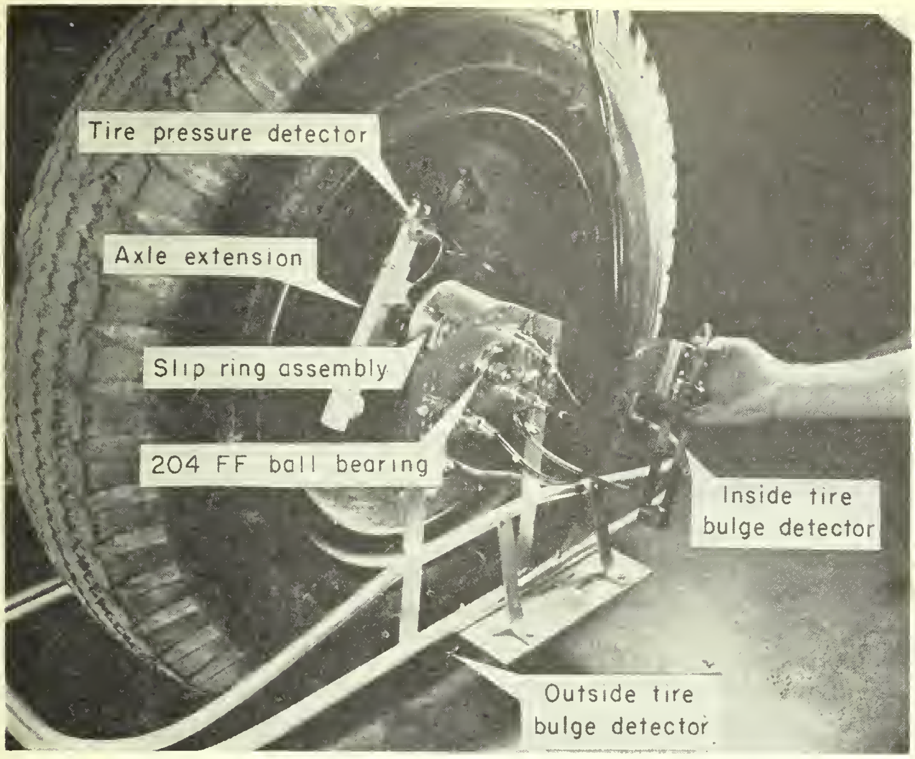
[4] *Vertical forces between the road surfaces and tires*, by Professor W. Endres and Dipl. Engineer J. Bombard. Institute for Testing Mechanical and Chemical Materials, Unter Den Eichen No. 87, Berlin-Dahlen, Germany. (Translated from the German by A. C. Benkelman, Bureau of Public Roads.)



Figures 3.—Instruments used to measure tire sidewall deflections and pressure changes.

of the sidewall would cause the spring steel straps to bend.

Strain gages were cemented to each side of both spring steel straps to provide a resistance bridge for measuring the amount of bend in the straps and thereby the bulge of the tire sidewall with increased load. The aluminum plate holding the steel straps was mounted vertically and bolted to an aluminum angle which was in turn bolted to a horizontal aluminum plate ¼ x 4 x 12 inches. Both the vertical and horizontal faces of the angle were slotted to permit an adjustment in the position of the roller with respect to the tire sidewall. A framework made by bending ¾-inch thin wall tubing was used to support the horizontal plate to which the sidewall bulge detector was attached. This plate was also slotted to permit a longitudinal adjustment. The framework of ¾-inch tubing was rectangular in shape with rounded corners and encircled the wheel.

A second framework of ½-inch tubing was placed above the ¾-inch framework. It had the same general shape as the ¾-inch framework but the sides were raised 4 inches at the center to make a more rigid framework. The two frameworks of tubing were brazed together at each end. This frame was then hung from the truck spring mounting bolts on the inside of the wheel and from a special axle extension on the outside of the wheel.

The axle extension was built from a broken truck axle and was long enough to extend about 3 inches beyond the plane of the tire sidewall as shown in figure 1. The face plates of the axle and the axle extension were placed back to back and held in position with the normal axle bolts. The end of the extension was trued with the axis rotation by means of a micrometer dial before the nuts were tightened. A 204FF ball bearing was placed near the end of the extension to provide a mounting for the plate on which the framework constructed of tubing was hung. Through this bearing, the framework was rigidly supported even though the axle extension rotated with the wheel. A sidewall bulge detector as previously described was placed on each side of the tire. Both were held in position by the rigid frame supported by the axle housing on the inside of the wheel and by the extension of the axle on the outside of the wheel, as shown in figure 2. Electrical cables from the gages on the bulge detectors were wired to sockets in the hanger plate to permit the cables leading to the recording equipment to be connected or disconnected with ease.

An initial bending moment was placed in the spring steel straps to obtain negative as well as positive readings for deflections of the sidewalls. The roller of the bulge detector was first adjusted so that it was in contact with the sidewall of the tire. A ½-inch steel block was then placed between the roller and the tire. After the resistance bridge circuit had been balanced to a zero reading, the block was removed and the bulge detector repositioned until a zero reading was again obtained on the recorder. This assured contact between the roller of the detector and the tire for all expected sidewall deflections.

A linear calibration curve, used to measure the deflection of the tire sidewalls, was obtained by a laboratory test calibration of the tire sidewall bulge detectors. A micrometer screw was utilized to move the roller of the detector through a one-inch range. A greater displacement was not tried because the original ½ inch plus or minus ½ inch was thought to be sufficient to record the expected range in the bulge of the sidewalls of the tire. This proved to be the case.

After the roller of the detector was in place and rolling along the tire sidewall as the truck wheel turned, a smooth path had to be provided on each side of the tire. Also, since the plane of the tire sidewall was not exactly perpendicular to the axis of rotation, that part of the tire in contact with the roller of the detector had to be trued. Otherwise, the wobble of the tire sidewall would have been recorded and could not have been distinguished from actual sidewall deflection due to a change in the load on the tire.

An emery wheel, placed in a fixed position while the jacked-up truck wheel was rotating, was used to provide a smooth and true path on each side of the tire for the roller of a detector. The largest amount of rubber removed at any one point was less than 1/16 inch. Some slight imperfections remained, however,

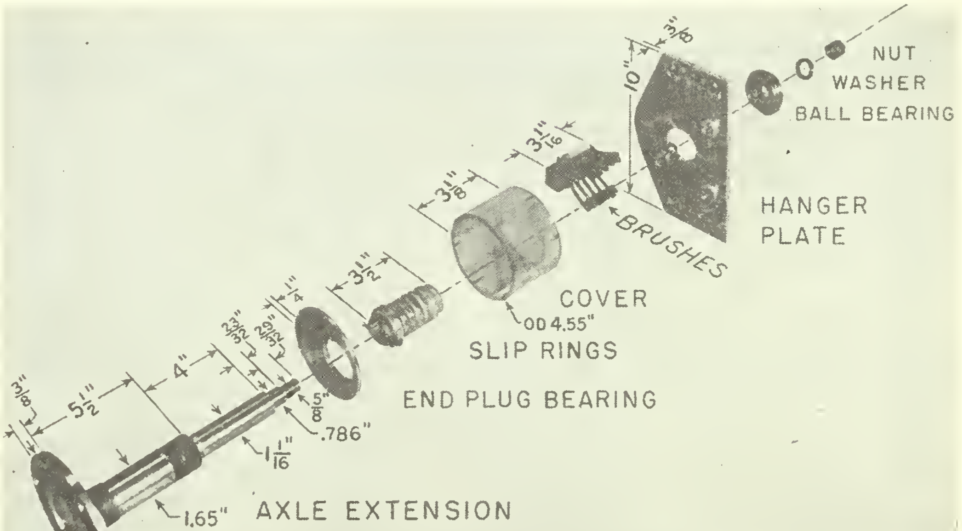


Figure 4.—Exploded view of the slipring assembly used to transfer the electrical circuits from the rotating wheel to the recording instruments installed on the truck bed.

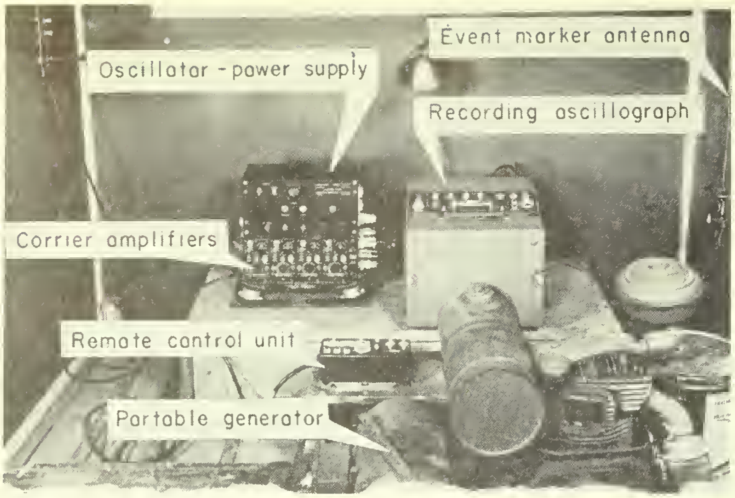


Figure 5.—Recording instruments and generator installed at the forward end of the truck bed.

and undoubtedly accounted for some error in the recordings.

Changes in Tire Air Pressure

The third method of measuring momentary changes in the load imparted to the tire was that of recording the changes in the air pressure within the tire. It was believed that the distortion of the tire casing at the point of contact with the pavement surface must cause a slight change in the volume of the inflated chamber of the tire. A change in volume would be accompanied by a change in pressure. The purpose of this phase of the study was to measure this change and calibrate its magnitude in terms of pounds of weight on the tire. There was considerable uncertainty as to whether the change would be great enough to measure accurately since no record could be found of any previous research on this phenomenon.

A Consolidated Electrodynamics Corp. pressure piekup gage, type 4–311,0–50 p. s. i. g., shown in figure 3, was used to record pressure changes. A hole was drilled in the valve stem of the tire tube and a short piece of ⅛-inch o. d. copper tubing was soldered into the valve stem. This tubing was connected to the pressure pickup which was mounted so that the axis of the diaphragm was parallel with the axis of rotation of the truck wheel. This minimized the effect of angular and vertical accelerations on the diaphragm. The tire was inflated to a pressure of 55 p. s. i. g. when cold.

The pressure pickup was an unbonded strain gage type and turned with the truck wheel. The electrical circuits, therefore, had to be transferred from the rotating wheel to the stationary bed of the truck. A slipring assembly of five rings, shown in figure 4, was built for this purpose. The rings were concentric with and fastened to the axle extension. They rotated with the truck wheel while the brushes remained stationary and were anchored to the framework provided for the bulge detectors. The brushes were wired to a socket in the hanger plate to permit an easy connection with the recording equipment in the truck. A plastic cover was provided for the slipring assembly to keep out dust and other foreign matter.

Calibration of the pressure pickup was accomplished by comparing the pressure pickup oscillogram with the maximum and minimum weights as recorded from the axle-housing strain and the tire sidewall deflection. It was not possible to make the calibration statically in the same manner as for the axle and sidewalls, because the galvanometer readings for the same air pressure were different with the sliprings stationary than when they were in rotation. The plotted points of comparison show some scatter due to normal imperfections in the sliprings and brushes. The use of high quality instrument-type sliprings would permit a more accurate calibration and would also make an oscillogram more representative of the actual magnitude of pressure change due to a change in the load on a tire.

Figure 5 shows the recording instruments for the three tests mounted on the forward end of the truck bed. A recording oscillograph with its standard oscillator, power supply, and carrier amplifier circuits was used to record the data. Power to operate this equipment was furnished by a portable generator which was placed on the truck bed just behind the instrument shelter. The shelter was built of

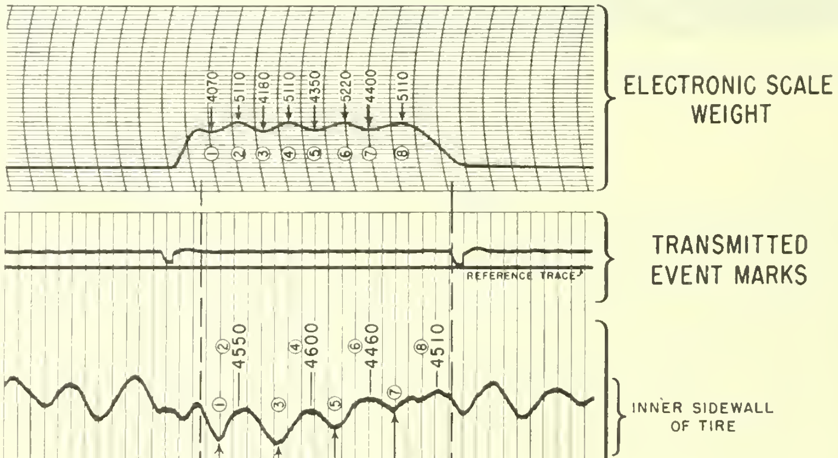


Figure 6.—Typical simultaneous oscillograms of wheel-load measurements registered by the electronic scale, axle-housing strain gages, tire sidewall detectors, and tire pressure pickup.

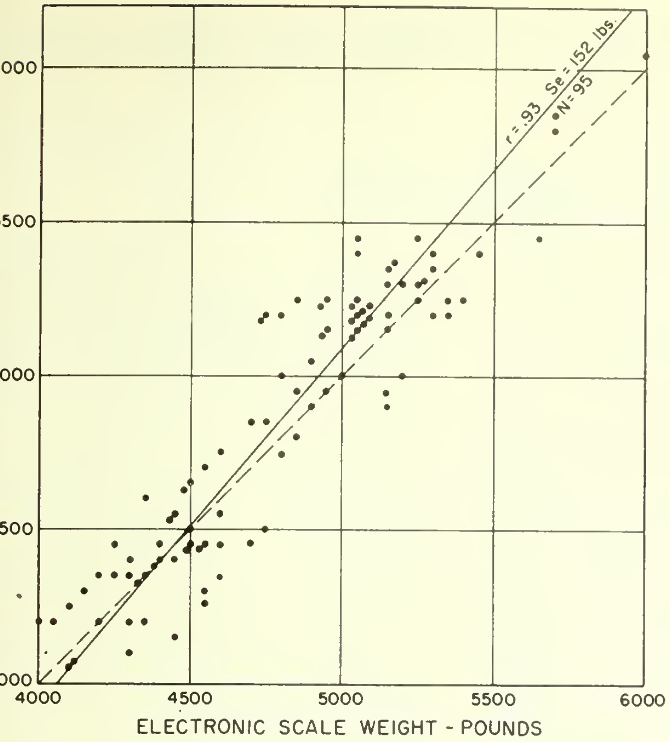


Figure 7.—Correlation between weights determined by the axle-housing strain gages and the electronic scale.

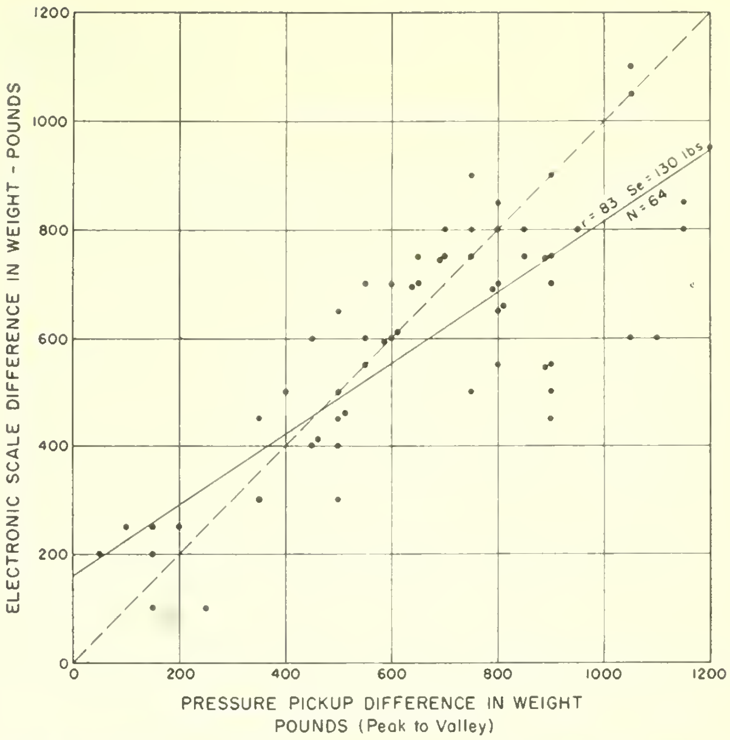


Figure 9.—Correlation between differences in weights recorded by the electronic scale and the pressure pickup.

3/16-inch tempered pressed-fiber panels which not only provided protection from the elements but also darkened the space as an aid to the operator in viewing the screen on the oscillograph. The rear panel of the instrument shelter was removed in figure 5 to permit a photographic view of the recording instruments.

All test runs included travel over the new electronic scale where simultaneous weight recordings were made for each run using a Brush amplifier and ink recorder. The Brush recorder was calibrated by placing known wheel and axle weights on the scale platform and recording these static weights. The static wheel and axle weights were obtained by weighing at the nearby Virginia weighing station. Only the right side of the vehicle passed over the platform since only the right rear wheel was instrumented.

In order to synchronize the recording of axle strain, tire bulge, and tire pressure with the wheel weight as recorded by the electronic scale, pneumatic detectors were placed at the beginning and end of the scale platform. The air switches of these detectors keyed a radio transmitter which was placed on the roadway shoulder as indicated in figure 1.

When a tire rolled over a pneumatic detector, the transmitter sent out a signal which was picked up by an antenna on the wall of the instrument shelter located on the truck. The signal was rectified by a germanium diode which had its output connected to one of the galvanometers in the oscillograph. This made a pip on the oscillogram each time a pneumatic detector was depressed by a tire. This marked the oscillogram of the wheel-load measurements so that it could be compared

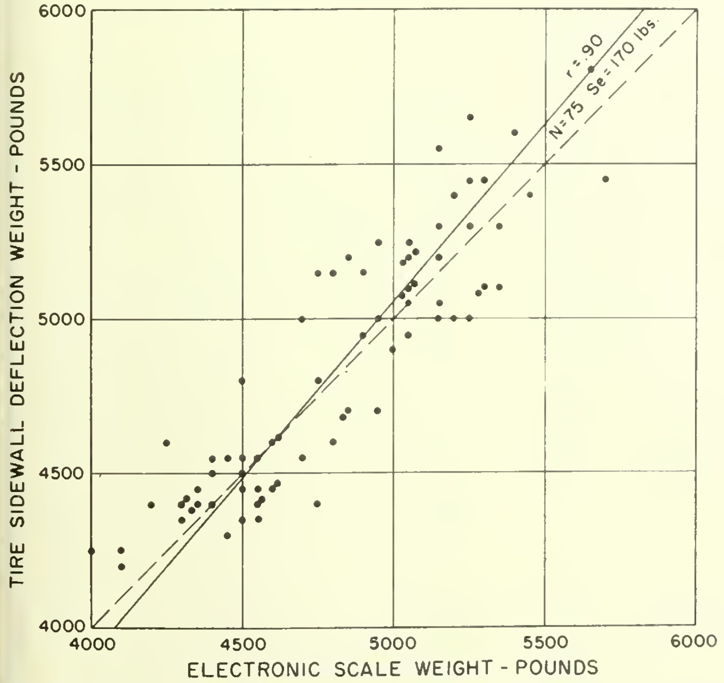


Figure 8.—Correlation between weights determined by the tire sidewall deflections and the electronic scale.

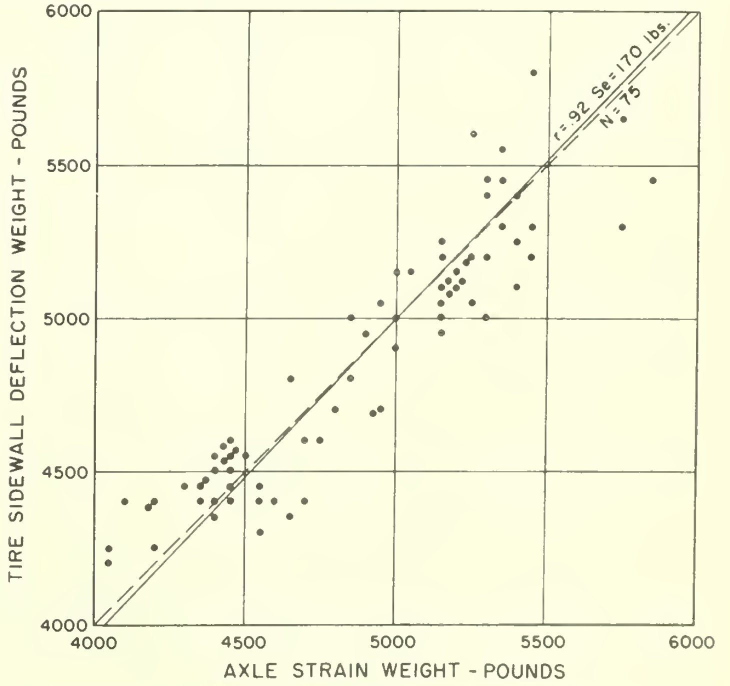


Figure 10.—Correlation between weights determined by the tire sidewall detectors and the axle-housing strain gages.

directly with the oscillogram of the electronic scale weight.

Oscillation of Load Artificially Increased

During trial runs across the scale platform it was found that the difference between static wheel weight and the dynamic wheel weight as well as the amplitude of the change in axle strain, tire bulge, and tire pressure was too small to obtain the desired comparison between the various trial methods of measuring the load on the tire. To obtain these desired differences and to establish definite points for comparison, the load was artificially oscillated at all speeds by a man jouncing up and down on the truck bed in rhythm with the natural frequency of the truck springs. This gave definite peaks and valleys for a direct comparison of the various methods. During analysis of the data an oscillogram peak for the instruments on the truck was compared with the simultaneous peak on the scale record, and similarly the valleys or low weights were compared. The additional weight of the man at the rear of the truck bed increased the static load on the right rear tire to about 4,750 pounds.

At creep speed the load was oscillated through 6 or 6½ cycles during the time the wheel was crossing the 7-foot scale platform. At 10 miles per hour, 1 or 1½ cycles were recorded; and at 20 miles per hour, about half a cycle was recorded. Speeds above 20 miles per hour were not included in this study because too many trips would have been necessary before a definite peak or valley was recorded on the electronic scale oscillogram. As speed appeared to have no effect within the range of the study, the data for all speed groups were combined for the analysis. Four trips each at creep speed and 5, 10, 15, and 20 miles per hour were made across the scale platform.

Figure 6 is a copy of typical simultaneous oscillograms from the electronic scale and the truck instruments. It shows the varying weight caused by the undulating vehicle load and the continuous wheel-load measurement by the three methods. The length of the scale platform is indicated on the wheel-load oscillogram by the event marker trace.

Comparison of Wheel-Load Measurements

The data taken from the oscillogram are plotted in figures 7–9 to show the correlation between the three methods of wheel-load measurement and the electronic scale. As a matter of interest the correlation is also shown between the weight as derived from axle-housing strains and that derived from tire sidewall deflections in figure 10. The static wheel load was 4,750 pounds. These data were recorded while the truck was artificially oscillated. The solid lines shown in figures 7–9 are the computed statistical regression lines which would be used to predict the weight recorded by the electronic scale from any of the other three recordings. The solid line in figure 10 is the relation between weights as shown by the axle-housing strain and the tire sidewall bulge.

The 45° broken line represents a hypothetical line of perfect correlation on which all plotted points would fall if there were no differences in the weights as indicated by the various methods. The regression lines would approach the ideal case as the weighing methods increase in accuracy. Correlation coefficients (r) and standard errors of estimate (Se) were calculated for each of these sets of data and appear on the graphs. The letter N on the graphs indicates the number of samples or observations.

It may be seen in figures 7 and 8 that the correlation is good. This indicates that the electronic scale does record the actual weight which a moving load imposes on the platform with reasonable accuracy.

From records taken while the truck was traveling over the open highway without the man producing artificial oscillations, it was found that an oscillation of from 2½ to 4 cycles per second did occur depending upon the roughness of the pavement surface. The lower frequency with increased amplitude was noted on the rougher surfaces and the higher frequencies occurred on the smoother surfaces.

Figure 9 shows that the correlation between the differences in weights as recorded by the pressure pickup and the electronic scale was less than for the other two methods used. This is due mostly to the changing circuit constant of the slipring assembly. A calibration of this random change showed that it was equal to about 225 pounds maximum, which would account for much of the observed error in this method.

Figure 10 shows that there was a high degree of correlation between the weights as recorded by the axle-housing strain and the tire sidewall deflection.

Three possibilities are being considered for improving the performance of the pressure pickup for future studies. The first is better quality sliprings. The second is a differential-type pressure pickup (5 p. s. i. differential) which should give a strong signal output compared to any slipring noise. The third is that of using a rotating pressure slip joint to eliminate the electrical sliprings between the pressure transducer and the recording equipment.

Tests of Concrete Containing Portland Blast-Furnace Slag Cement

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by WILLIAM E. GRIEB and GEORGE WERNER,
Highway Physical Research Engineers

Portland blast-furnace slag cement, a relatively new product in this country, is being used increasingly as an alternate for portland cement. Because of this fact there is need for greater knowledge of its strength and durability.

This article discusses the preliminary results of research undertaken by the Bureau's laboratory. The tests reported here are being continued, and it is contemplated that a final report will be prepared in 1958.

In general, the tests show good results for the portland blast-furnace slag cements. Air-entrained concretes containing portland blast-furnace slag cements give lower compressive and flexural strengths at curing ages of 3 and 7 days, equal strengths at 28 days, and higher strengths at 90 days than comparable concretes containing portland cements.

Varying the water-cement ratios of concretes containing portland blast-furnace slag cement or portland cement had only a limited effect on the relations of their compressive and flexural strengths. Tests to determine shrinkages due to loss of moisture indicated about the same results for the two types of cement.

THE INCREASED USE of portland blast-furnace slag cement in concrete for highway and bridge construction as an alternate for type I portland cement has interested engineers in the strength and durability of concrete made with a cement that is relatively new in the United States. Although considerable data have been published by Europeans on the properties of slag or portland blast-furnace slag cements, little has been published in this country.

Specification M 151–53 of the American Association of State Highway Officials defines portland blast-furnace slag cement as an intimately interground mixture of type I portland cement clinker and granulated blast-furnace slag and limits the slag content to a range of 25 to 65 percent of the final product.[2] The specification also gives requirements for the chemical composition of the cement and the slag constituent, together with requirements for physical tests of neat cement and mortar. The requirements for compressive strengths of mortar are identical with those for type I portland cement as given in AASHO specification M 85–53. As the slag does not require burning in a kiln, the output of a portland cement mill can be increased through the production of this cement without the use of additional equipment.

Information concerning the strength, durability, and resistance to attack by chloride salts of concrete prepared with portland blast-furnace slag cement is needed. Some data, now available, indicate that the strengths at early ages of this concrete are lower than those of concretes containing portland cement, but little information on the behavior of these concretes at advanced ages can be found. Values for the modulus of elasticity are also needed, particularly if portland blast-furnace slag cement is used in prestressed concrete.

This article presents data on the physical properties of air-entraining concretes prepared with type IS portland blast-furnace slag cements and type I portland cements from each of five cement plants. In each case, the clinker used in preparing the portland blast-furnace slag cement and the portland cement was the same. Compressive and flexural strengths of moist-cured concretes prepared with 5½, 6½, and 7½ gallons of water per bag of cement are given for ages ranging from 3 to 90 days, with values for the sonic modulus of elasticity for most of these ages. Strengths are also given for 5½ gallons per sack mixes tested at an age of 28 days after either normal or intermittent curing. Shrinkage tests on mixes containing 5½ and 6½ gallons per sack are reported for two conditions of initial curing followed by intermittent curing. Freezing and thawing tests in the laboratory on mixes containing 5½ and 6½ gallons per sack are reported up to 100 cycles and 50 cycles, respectively.

The strength and modulus of elasticity tests will be continued to an age of 1 year. Tests for shrinkage and resistance to freezing and thawing will be continued until conditions warrant their termination. Other tests to determine the resistance of the concretes to scaling caused by calcium chloride in removing ice are in progress but no results are now available. Strength tests will be repeated on specimens prepared with concrete containing 4½ gallons of water per bag of cement.

Conclusions

In general, the results of these tests show that air-entraining concretes containing portland blast-furnace slag cements, type IS, give lower compressive and flexural strengths at ages of 3 and 7 days, equal strengths at 28 days, and higher strengths at 90 days than comparable concretes containing portland cements, type I.

Detailed conclusions for air-entraining concretes with water-cement ratios of 5½, 6½, and 7½ gallons of water per sack of cement and containing portland blast-furnace slag cement, type IS, or portland cement, type I, follow:

1. Four of the five portland blast-furnace slag cements gave lower compressive and flexural strengths in moist-cured concretes at 3 and 7 days, equal strengths at 28 days, and higher strengths at 90 days than portland cements used in comparable concretes.
2. One portland blast-furnace slag cement gave lower compressive strengths in moist-cured concretes at 3, 7, 28, and 90 days than the portland cement used in comparable concretes. Flexural strengths were lower at 3, 7, and 28 days, but higher at 90 days.
3. Intermittent curing caused greater reductions in strength at 28 days for concretes containing portland blast-furnace slag cements than for concretes containing portland cements.
4. The relations between the strengths of concretes prepared with the two types of cement were not affected appreciably by changes in the water-cement ratio.
5. Shrinkages of portland blast-furnace slag cement concretes due to loss of moisture were the same as those for portland cement concretes.
6. Sonic moduli of elasticity for portland blast-furnace slag cement concretes were slightly lower at ages of 3, 7, and 28 days and higher at 90 days than the values for comparable portland cement concretes.
7. The amount of air-entraining solution needed to produce 5½ percent of air in the concretes varied for the 5 portland blast-furnace slag cements from approximately the same amount as used with the portland cements to about 2½ times that quantity.

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[1] This article was presented at the 36th Annual Meeting of the Highway Research Board, Washington, D. C., January 1957.

[2] Tests were made by the producers of portland blast-furnace slag cement to determine the optimum amount of slag. Approximately 40 percent of slag was used in the cements included in these tests.

Table 1.—Chemical composition [1] and physical properties of cements

	Source A cement		Source B cement		Source C cement		Source D cement		Source E cement	
	Type I	Type IS	Type I	Type IS	Type I	Type IS	Type I	Type IS	Type I	Type IS
Chemical composition (percent):										
Silicon dioxide	20.2	25.0	20.8	26.2	21.1	25.5	21.0	26.7	21.3	26.1
Aluminum oxide [2]	6.4	9.2	5.4	8.0	5.4	7.1	5.8	7.8	5.9	7.2
Ferric oxide	2.8	2.1	3.1	2.3	2.1	1.7	3.3	2.9	3.5	2.4
Calcium oxide	63.4	53.0	63.6	53.6	65.5	59.2	63.4	54.6	62.5	55.7
Magnesium oxide	2.7	3.7	2.8	3.6	2.4	1.8	2.2	3.3	1.7	2.8
Sulfur trioxide	2.3	2.0	1.8	2.3	1.3	1.4	1.9	2.3	2.3	2.2
Loss on ignition [3]	1.5	1.6	1.6	.8	1.4	1.3	1.2	.04	1.3	1.7
Sodium oxide	.8	.12	.09	.09	.04	.07	.10	.10	.15	.15
Potassium oxide	.36	.38	.15	.20	.43	.82	.17	.18	1.02	.91
Equivalent alkalies as Na_2O	.32	.37	.19	.22	.32	.61	.21	.22	.82	.75
Manganic oxide	.53	.87	.51	.85	.03	.08	.29	.32	.07	.50
Sulfide sulfur	.09	.56	.10	.68	.01	.33	.08	1.04	.03	.40
Insoluble residue	.28	.57	.16	.73	.22	.98	.08	.20	.20	.26
Chloroform-soluble organic substances	.003	.005	.003	.006	.003	.002	.003	.005	.003	.008
Calculated compounds (percent):										
Tricalcium silicate	51	--------	55	--------	63	--------	49	--------	41	--------
Dicalcium silicate	20	--------	18	--------	13	--------	23	--------	30	--------
Tricalcium aluminate	12	--------	9	--------	11	--------	10	--------	10	--------
Tetracalcium aluminoferrite	8	--------	9	--------	6	--------	10	--------	11	--------
Calcium sulfate	3.9	--------	3.0	--------	2.2	--------	3.2	--------	3.9	--------
Merriman sugar test:										
Neutral point ml	31.4	3.4	17.7	6.9	46.2	19.2	5.5	2.2	8.3	4.2
Clear point ml	44.9	4.0	23.3	8.0	68.0	26.2	6.2	2.2	9.7	5.3
Physical properties:										
Apparent specific gravity	3.14	2.99	3.12	3.03	3.12	3.05	3.14	3.03	3.14	3.03
Specific surface (Blaine) cm.²/g	3,810	5,000	3,325	4,820	3,425	3,775	3,680	4,040	3,555	3,605
Passing No. 325 sieve percent	92.4	96.0	87.5	97.6	80.0	82.9	94.4	98.2	96.2	94.9
Autoclave expansion do	.10	.01	.06	.00	.09	.04	.07	−.02	.04	.00
Normal consistency do	25.2	27.6	23.8	27.2	20.4	22.6	26.6	31.2	26.5	26.6
Time of setting (Gillmore):										
Initial hours	2.9	2.8	3.7	3.4	1.7	2.6	4.2	3.5	2.9	2.8
Final do	5.1	6.8	5.6	6.2	3.4	3.9	6.5	7.9	4.8	6.2
Compressive strength (1:2.75 mortar):										
At 3 days p. s. i.	2,700	2,555	1,620	1,730	1,960	1,740	2,635	1,740	2,465	1,525
At 7 days p. s. i.	3,860	4,170	2,625	2,630	3,440	2,940	3,940	2,750	3,525	2,360
At 28 days p. s. i.	5,645	6,120	4,150	5,190	5,145	5,120	5,615	4,615	4,965	3,900
Tensile strength (1:3 mortar):										
At 3 days p. s. i.	295	290	280	265	265	240	305	265	320	245
At 7 days p. s. i.	365	420	355	335	360	345	390	340	375	315
At 28 days p. s. i.	450	500	450	445	415	425	420	460	435	455
Mortar air content percent	11.6	8.4	10.4	7.4	8.4	8.1	10.7	8.3	9.4	9.2

[1] All determinations made in accordance with current ASTM methods for portland cement.
[2] Values for aluminum oxide include any titanium and phosphorous oxides that may be present.
[3] Values for type IS cements do not represent accurately the volatile matter. Oxidation of sulfide sulfur during the test affects these results.

Materials and Proportions

In order to avoid repetition of the terms "portland blast-furnace slag cement, type IS," and "portland cement, type I," they are hereafter referred to as slag cement [3] and portland cement, respectively.

The five slag cements used in these tests were obtained from two mills of one producer and from one mill of each of three producers. Portland cements ground from the same clinkers as those used to prepare the slag cements were also supplied for the study. Chemical analyses and physical properties of all cements are given in table 1. Values for free lime are not shown because determinations on all cements had not been completed at the time of this report. Time of setting by the Vicat test method is not given for the slag cements but the values obtained were within the specified limits of AASHO specification M 151.

The grading and physical properties of the fine and coarse aggregates used are given in table 2. Mix data for all concretes are given in table 3. The mixes were designed on a water-cement ratio basis for air-entrained concrete of 5½ percent air, 2- to 3-inch slump, and a b/b_o of 0.72.[4] The cement contents were approximately 6.1, 4.9, and 4.1 bags per cubic yard of concrete for water contents of 5½, 6½, and 7½ gallons of water per bag of cement. An exception occurred in the mix containing 6½ gallons of water for slag cement from source D; a cement content of 5.1 bags was necessary to maintain a slump of 2 inches. It is noted also that the slumps of concretes containing 5½ and 7½ gallons of water with this cement were less than 2 inches but no adjustments in proportions were made. Air was entrained in the concrete by use of a commercial solution of neutralized Vinsol resin.

Table 2.—Grading and physical properties of the aggregates

Grading and physical properties	White Marsh sand	Riverton limestone
GRADING OF AGGREGATES—PERCENTAGE PASSING SIEVES		
Sieve size		
1-inch	------	100
¾-inch	------	70
½-inch	------	40
⅜-inch	100	24
No. 4	96	0
No. 8	79	------
No. 16	66	------
No. 30	50	------
No. 50	23	------
No. 100	7	------
Fineness modulus	2.79	7.06
PHYSICAL PROPERTIES OF AGGREGATES		
Bulk specific gravity:		
Dry	2.63	2.78
Saturated surface dry	2.65	2.79
Absorption percent	0.4	0.4
Strength ratio (Ottawa sand):		
Compressive strength:		
At 7 days percent	158	------
At 28 days do	167	------
Tensile strength:		
At 7 days do	106	------
At 28 days do	116	------
Los Angeles wear test, grading A, loss percent	------	20.2
Accelerated soundness, Na_2SO_4, loss percent	------	3.5

Mixing, Molding, and Curing Specimens

All mixing was done in an open pan-type laboratory mixer of 1¾-cubic foot capacity. Both fine and coarse aggregates were used in a wet condition and the amount of mixing water needed for each batch of concrete was corrected for the free water in the aggregates. The following mixing procedure was employed: The cement and the wet sand were mixed for 30 seconds and part of the required amount of water and all of the air-entraining solution were then added and mixed for 30 seconds. The wet coarse aggregate was then added with the necessary amount of additional water. The concrete was mixed for 2½ minutes after the addition of the coarse aggregate. Determinations of the slump and unit weight and air content by the pressure method were made. The portions of the concrete used for the slump and unit weight tests were returned to the concrete in the mixer and remixed for 15 seconds before the test specimens were prepared.

All beams were made in individual molds. Beams and cylinders that were to be tested after continuous moist curing were molded, cured, and tested in accordance with applicable AASHO procedures.

Specimens for outdoor exposure to determine the resistance of the concrete to scaling

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[3] Portland blast-furnace slag cement is not to be confused with slag cement which is defined in ASTM tentative specification C358-55 as a finely divided material consisting of water-quenched granulated blast-furnace slag and hydrated lime.

[4] The term b/b_o is defined as "the dry, rodded volume of coarse aggregate in a unit volume of concrete" in the article *A Method for Proportioning Concrete for Compressive Strength, Durability and Workability*, by A. T. Goldbeck and J. E. Gray. National Crushed Stone Association, Washington, D. C., Bulletin No. 11, Dec. 1942, p. 10.

Table 3.—Mix data for laboratory specimens

Cement		5½ gallons of water per sack of cement [1]				6½ gallons of water per sack of cement [1]				7½ gallons of water per sack of cement [1]			
Source	Type	Cement content	Slump	Air	Vinsol resin added	Cement content	Slump	Air	Vinsol resin added	Cement content	Slump	Air	Vinsol resin added
		Sacks/ cu. yd.	*Inches*	*Percent*	*Ml./sack*	*Sacks/ cu. yd.*	*Inches*	*Percent*	*Ml./sack*	*Sacks/ cu. yd.*	*Inches*	*Percent*	*Ml./sack*
A	I	6.07	2.9	5.5	18.8	4.94	2.6	5.4	20.6	4.15	2.1	5.7	20.8
A	IS	6.06	2.8	5.3	25.2	4.93	2.7	5.6	32.3	4.12	2.3	5.6	32.3
B	I	6.06	2.9	5.6	15.8	4.93	2.4	5.6	19.7	4.15	2.2	5.4	19.4
B	IS	6.04	2.7	5.2	38.2	4.91	2.6	5.5	48.3	4.14	2.3	5.2	49.3
C	I	6.09	3.3	5.4	18.1	4.94	2.4	5.5	20.7	4.16	2.1	5.4	23.2
C	IS	6.07	3.2	5.5	17.8	4.93	2.2	5.5	22.0	4.14	2.3	5.7	23.2
D	I	6.07	2.3	5.6	17.7	4.95	2.1	5.4	21.0	4.14	1.9	5.6	21.4
D	IS	6.06	1.3	5.5	26.5	[2] 5.14	2.0	5.4	29.2	4.15	1.6	5.7	30.7
E	I	6.09	2.8	5.4	19.3	4.94	2.2	5.5	22.2	4.15	2.0	5.4	23.9
E	IS	6.06	2.2	5.5	21.6	4.94	2.0	5.3	22.8	4.15	1.8	5.5	24.8

[1] Proportions by oven-dry weight except as noted: 5½ gal. mix=94-190-315, 6½ gal. mix=94-255-390, and 7½ gal. mix=94-325-460. (Each value in table is an average of 5 tests.)
[2] Proportions by oven-dry weight: 94-240-375.

caused by freezing and the removal of ice by calcium chloride were slabs 16 by 24 by 4 inches. A raised edge was cast around the perimeter of each slab. After being moist cured for 28 days under standard conditions, the slabs were placed in the exposure plot for freezing.

Beams for freezing and thawing in water by the slow cycle procedure described in ASTM Method C 292 were moist cured for 7 days, cured in laboratory air at 72° F. and 50 percent relative humidity for 14 days, and immersed in water for 7 days at 72° F.

Cylinders and beams that were cured intermittently for strength tests and beams that were cured intermittently for shrinkage tests were stored in moist air or water at 72° F. and in laboratory air at 72° F. and 50 percent relative humidity.

Discussion of Test Results

The results of tests for strength of continuously moist-cured specimens and of intermittently cured specimens are shown in tables 4–7. Data for sonic modulus of elasticity are given in table 8, and a comparison of the sonic and static moduli is given in table 9. Data for freezing and thawing tests are shown in table 10. Average values for strengths are shown in figures 1 and 2 and for shrinkage tests in figures 3 and 4. These data are discussed mainly by comparing the results of tests of concretes containing slag cement with those of concretes containing portland cement.

Strengths of Moist-Cured Specimens

The average compressive strengths of concretes given in table 4 show that the concretes containing slag cements from sources A, B, C, and D were about the same or greater at ages of 28 and 90 days than the strengths of corresponding concretes prepared with portland cements. In general, the strengths of the slag cement concretes were less than the corresponding portland cement concretes at ages of 3 and 7 days. In no case did the strengths of concrete made with slag cement E equal the strengths of concrete made with portland cement from the same source. With few exceptions, the strengths of concretes made with slag cement E did not equal those of corresponding concretes containing slag cements from any of the other sources.

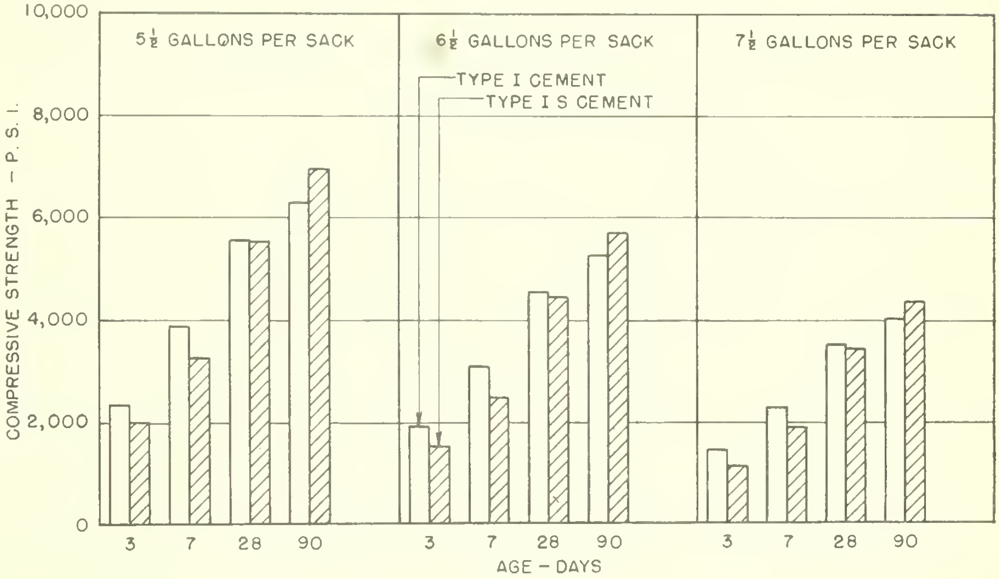

Figure 1.—Comparison of compressive strengths of concrete containing portland cements (type I) and portland blast-furnace slag cements (type IS) from 5 sources.

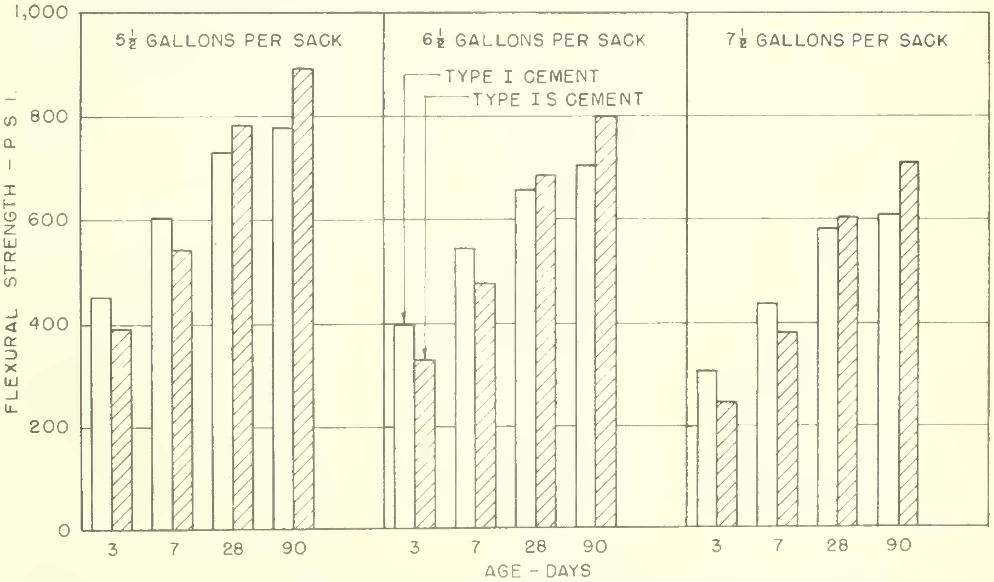

Figure 2.—Comparison of flexural strengths of concretes containing portland cements (type I) and portland blast-furnace slag cements (type IS) from 5 sources.

The average flexural strengths given in table 5 for all slag cement concretes are, in general, lower at 3 and 7 days and higher at 28 and 90 days than the strengths of the corre-

Table 4.—Compressive strength tests on moist-cured specimens [1]

Cement		5½ gallons of water per sack of cement. Compressive strength after—				6½ gallons of water per sack of cement. Compressive strength after—				7½ gallons of water per sack of cement. Compressive strength after—			
Source	Type	3 days	7 days	28 days	90 days	3 days	7 days	28 days	90 days	3 days	7 days	28 days	90 days
		P. s. i.	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*
A	I	2, 610	4, 030	5, 620	6, 440	2, 050	3, 260	4, 670	5, 440	1, 500	2, 350	3, 710	4, 130
A	IS	2, 340 (90)	3, 810 (95)	5, 940 (106)	7, 140 (111)	1, 770 (86)	3, 060 (94)	4, 960 (106)	6, 170 (113)	1, 320 (87)	2, 250 (96)	3, 780 (102)	4, 720 114)
B	I	1, 940	3, 330	5, 400	6, 380	1, 510	2, 580	4, 180	5, 190	1, 000	1.740	2, 980	3, 620
B	IS	1, 940 (100)	3, 020 (91)	5, 790 (107)	7, 640 (120)	1, 440 (95)	2, 310 (90)	4, 560 (109)	6, 460 (124)	1, 070 (107)	1, 770 (102)	3, 700 (124)	4, 970 (137)
C	I	2, 070	3, 710	5, 070	5, 500	1, 770	2, 950	4, 100	4, 650	1, 320	2, 240	3, 250	3, 640
C	IS	1, 720 (83)	3, 220 (87)	5, 320 (105)	6, 390 (116)	1, 360 (77)	2, 430 (82)	4, 240 (102)	5, 030 (108)	1, 010 (77)	1, 810 (81)	3, 160 (97)	3, 870 (106)
D	I	2, 650	4, 160	6, 050	6, 920	2, 180	3, 400	4, 970	5, 780	1, 600	2, 350	3. 810	4, 580
D	IS	2, 110 (80)	3, 390 (81)	6, 150 (102)	7, 600 (110)	1, 630 (75)	2, 440 (72)	4, 860 (98)	6, 110 (106)	1, 230 (77)	2, 120 (90)	3, 670 (96)	4, 720 (103)
E	I	2, 680	4, 150	5, 660	6, 240	2, 200	3, 310	4, 790	5, 320	1, 750	2, 760	3, 860	4, 150
E	IS	1, 830 (68)	2, 790 (67)	4, 460 (79)	5, 900 (95)	1, 350 (61)	2, 220 (67)	3, 680 (77)	4, 770 (90)	1, 010 (58)	1, 620 (59)	2. 780 (72)	3, 600 (87)
Average, type I		2, 390	3, 880	5, 560	6, 300	1, 940	3, 100	4, 540	5, 280	1, 430	2, 290	3, 520	4, 020
Average, type IS		1, 990 (83)	3, 250 (84)	5, 530 (99)	6, 930 (110)	1, 510 (78)	2, 490 (80)	4, 460 (98)	5, 710 (108)	1, 130 (79)	1, 910 (83)	3, 420 (97)	4, 380 (109)

[1] Figures in parentheses represent ratios (in percent) of the strength of the concrete containing type IS cement to the strength of the corresponding concrete containing type I cement. Each value is an average of 5 tests. Specimens, capped with neat Lumnite cement, were 6- by 12-inch cylinders stored in moist air until tested.

Table 5.—Flexural strength tests on moist-cured specimens [1]

Cement		5½ gallons of water per sack of cement. Flexural strength after—				6½ gallons of water per sack of cement. Flexural strength after—				7½ gallons of water per sack of cement. Flexural strength after—			
Source	Type	3 days	7 days	28 days	90 days	3 days	7 days	28 days	90 days	3 days	7 days	28 days	90 days
		P. s. i.	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*
A	I	495	605	730	760	420	560	635	695	315	425	590	605
A	IS	420 (85)	610 (101)	880 (121)	920 (121)	365 (87)	535 (96)	715 (113)	800 (115)	270 (86)	435 (102)	660 (112)	730 (121)
B	I	380	570	740	790	330	485	695	760	240	370	520	645
B	IS	395 (104)	520 (91)	750 (101)	885 (112)	305 (92)	440 (91)	690 (99)	800 (105)	235 (98)	365 (99)	640 (123)	780 (121)
C	I	410	550	630	695	345	505	565	600	270	430	530	530
C	IS	365 (89)	550 (100)	730 (116)	815 (117)	315 (91)	475 (94)	690 (122)	775 (129)	220 (81)	355 (83)	555 (105)	630 (119)
D	I	500	610	750	780	435	555	640	720	335	455	600	625
D	IS	400 (80)	525 (86)	775 (103)	925 (119)	345 (79)	485 (87)	730 (114)	835 (116)	275 (82)	395 (82)	630 (105)	760 (122)
E	I	495	670	805	855	440	620	730	745	355	490	670	635
E	IS	375 (76)	505 (75)	790 (98)	915 (107)	310 (70)	445 (72)	610 (84)	780 (105)	225 (63)	340 (69)	540 (81)	650 (102)
Average, type I		455	600	730	775	395	545	655	705	305	435	680	610
Average, type IS		390 (86)	540 (90)	785 (108)	890 (115)	330 (84)	475 (87)	685 (105)	800 (113)	245 (80)	380 (87)	605 (104)	710 (116)

[1] Figures in parentheses represent ratios (in percent) of the strength of concrete containing type IS cement to the strength of the corresponding concrete containing type I cement. Each value is an average of 5 tests. Specimens, stored in moist air until tested, were 6- by 6- by 21-inch beams tested in accordance with ASTM Method C-78 with third point loading on an 18-inch span—side as molded in tension.

sponding portland cement concretes. The superior strengths of concretes made with slag cement B as found in the compressive strength tests do not appear in the results for flexural strength. Slag cement from source E, however, showed better relative strengths in the tests for flexural strength than for compressive strength. The concretes prepared with slag cement E had lower compressive strengths than those prepared with portland cement E at all ages, but the flexural strengths obtained with the slag cement were higher than those for portland cement only at an age of 90 days.

Effect of Intermittent Curing on Strength

In table 6, comparisons are given showing the effect of intermittent or partial curing on the compressive strength of concrete. One group of specimens prepared with each cement was moist cured for 28 days. Other groups were moist cured for 1 or 7 days, followed by curing in laboratory air. Two groups were immersed in water for 24 hours immediately prior to testing, and one group was tested in an air-dry condition. As was expected, the specimens moist cured for 8 days or less failed to develop the strength of the specimens moist cured continuously. The specimens prepared with portland cement usually showed less reduction in strength than those containing slag cement, but the difference is not marked.

Similar tests for flexural strength are shown in table 7. These data show the same trend as found in tests for compressive strength in that the concrete specimens prepared with portland cement show less reduction in strength due to the intermittent or partial curing. However, 4 of 5 concretes prepared with portland cement and one prepared with

Table 6.—Compressive strength tests on specimens cured intermittently [1]

Cement		5½ gallons of water per sack of cement. Compressive strength after curing for—			
Source	Type	1 day moist, 27 days dry [2]	1 day moist, 26 days dry,[2] 1 day soak	7 days moist, 20 days dry,[2] 1 day soak	28 days moist [3]
		P. s. i.	*P. s. i.*	*P. s. i.*	*P. s. i.*
A	I	4, 420 (77)	3, 980 (69)	5, 100 (88)	5, 770
A	IS	4, 080 (69)	3, 760 (63)	5, 360 (90)	5, 940
B	I	3, 420 (63)	3, 050 (56)	4, 660 (86)	5, 400
B	IS	4, 000 (67)	3. 270 (55)	4, 790 (81)	5, 950
C	I	3, 320 (64)	3, 030 (59)	4, 720 (91)	5, 160
C	IS	3, 180 (61)	2, 840 (55)	4, 490 (86)	5, 200
D	I	4, 140 (72)	3, 620 (62)	5, 260 (91)	5, 790
D	IS	4, 050 (69)	3, 450 (59)	5, 010 (86)	5, 830
E	I	4, 040 (75)	3, 820 (70)	4, 960 (92)	5, 420
E	IS	3, 700 (80)	2, 870 (62)	4, 180 (91)	4, 600
Average, type I		3, 870 (70)	3, 500 (64)	4, 940 (90)	5, 510
Average, type IS		3, 800 (69)	3, 240 (59)	4, 770 (87)	5, 500

[1] Figures in parentheses represent the ratios (in percent) of the strength of the intermittently cured specimens (6- by 12-inch cylinders) to the strength of the 28-day moist-cured specimens. Each value is an average of 5 tests.
[2] Specimens capped with sulfur cement.
[3] Specimens capped with neat Lumnite cement.

Table 7.—Flexural strength tests on specimens cured intermittently [1]

Cement		5½ gallons of water per sack of cement. Flexural strength after curing for—			
Source	Type	1 day moist, 27 days dry	1 day moist, 26 days dry, 1 day soak	7 days moist, 20 days dry, 1 day soak	28 days moist
		P. s. i.	*P. s. i.*	*P. s. i.*	*P. s. i.*
A	I	435 (58)	565 (76)	715 (96)	745
A	IS	470 (54)	585 (67)	730 (83)	875
B	I	445 (63)	425 (60)	750 (106)	710
B	IS	470 (63)	515 (69)	785 (105)	745
C	I	445 (63)	420 (59)	710 (100)	710
C	IS	395 (51)	410 (53)	730 (95)	770
D	I	485 (64)	570 (75)	765 (101)	760
D	IS	490 (59)	465 (56)	815 (99)	825
E	I	480 (63)	570 (75)	785 (103)	765
E	IS	435 (55)	465 (59)	720 (91)	790
Average, type I		460 (62)	510 (69)	745 (101)	740
Average, type IS		450 (56)	490 (61)	755 (94)	800

[1] Figures in parentheses represent the ratios (in percent) of the strength of the intermittently cured specimens to the strength of the 28-day moist-cured specimens. Each value is an average of 5 tests. Specimens were 6- by 6- by 21-inch beams tested in accordance with ASTM Method C 78 with third point loading on an 18-inch span—side as molded in tension.

Table 9.—Comparison of sonic and static moduli of elasticity for concretes containing 7½ gallons of water per sack of cement and tested at 90 days

Cement		Sonic modulus of elasticity	Static modulus of elasticity
Source	Type		
		P. s. i. x 10^6	*P. s. i.* x 10^6
A	I	6. 07	5. 57
A	IS	6. 16	5. 90
B	I	6. 00	5. 42
B	IS	6. 27	6. 10
C	I	5. 98	5. 26
C	IS	6. 20	5. 60
D	I	6. 04	5. 90
D	IS	6. 33	6. 44
E	I	6. 19	5. 58
E	IS	6. 11	5. 64
Average, type I		6. 06	5. 55
Average, type IS		6. 21	5. 94

Table 8.—Sonic modulus of elasticity [1]

Cement		Sonic modulus of elasticity—pounds per square inch x 10^6													
		5½ gallons of water per sack of cement					6½ gallons of water per sack of cement					7½ gallons of water per sack of cement			
Source	Type	3 days	7 days	28 days	90 days	180 days	3 days	7 days	28 days	90 days	180 days	3 days	7 days	28 days	90 days
A	I	------	------	6. 38	6. 76	6. 82	4. 95	5. 73	6. 32	6. 66	6. 80	4. 18	5. 22	5. 80	6. 07
A	IS	------	------	6. 17	6. 70	6. 84	4. 79	5. 45	6. 04	6. 71	6. 92	3. 85	4. 86	5. 72	6. 16
B	I	----	5. 70	6. 46	6. 52	6. 62	4. 44	5. 44	6. 19	6. 57	6. 70	3. 66	4. 84	5. 62	6. 00
B	IS	------	4. 95	6. 11	6. 72	6. 91	4. 26	4. 97	5. 86	6. 41	6. 71	3. 59	4. 61	5. 51	6. 27
C	I	------	5. 75	6. 33	6. 55	6. 69	4. 62	5. 57	6. 15	6. 33	6. 46	3. 99	5. 06	5. 83	5. 98
C	IS	------	5. 65	6. 39	6. 57	6. 67	4. 38	5. 42	6. 20	6. 57	6. 76	3. 65	4. 72	5. 73	6. 20
D	I	------	6. 08	6. 43	6. 69	6. 75	4. 91	5. 69	6. 27	6. 59	6. 72	4. 36	5. 19	5. 89	6. 04
D	IS	------	5. 95	6. 65	7. 01	7. 18	4. 66	5. 35	6. 22	6. 63	6. 89	4. 00	4. 76	5. 82	6. 33
E	I	------	6. 20	6. 65	6. 78	6. 95	5. 06	5. 84	6. 45	6. 69	6. 77	4. 56	5. 51	6. 15	6. 19
E	IS	------	5. 57	6. 27	6. 54	6. 91	4. 58	5. 30	5. 97	6. 59	6. 81	3. 94	4. 73	5. 67	6. 11
Average, type I		------	5. 93	6. 45	6. 65	6. 77	4. 80	5. 65	6. 28	6. 57	6. 69	4. 15	5. 16	5. 86	6. 06
Average, type IS		------	5. 53	6. 32	6. 71	6. 90	4. 53	5. 30	6. 06	6. 58	6. 82	3. 81	4. 74	5. 69	6. 21

[1] Sonic modulus determined on 6- by 6- by 21-inch beams prior to testing for flexural strength. Specimens were continuously moist cured. Each value is an average of 5 or more tests.

slag cement developed as much or more strength when given the initial 7-day moist curing with 24 hours immersion in water prior to testing as when moist cured continuously. It is possible that these results were caused by the so-called "shell effect," where only the outer portion of the beam was wetted by the 24-hour immersion treatment before testing and the major portion of the beam was relatively dry. As failure to saturate the specimen thoroughly affects the flexural strength in many ways, it is unwise to consider that only 7 days initial moist curing for concrete is adequate for concrete tested under laboratory conditions. It should be emphasized that only one water-cement ratio (5½ gallons of water per sack of cement) was used in these intermittent curing tests for strength. The effect of this curing on mixes of higher or lower water-cement ratio might furnish somewhat different data.

Sonic Modulus of Elasticity

The data given in table 8 for sonic modulus of elasticity, in general, show the same trends as the tests for compressive strength. At

Table 10.—Results of freezing and thawing tests [1]

Cement		Percent of original N^2 after freezing and thawing for indicated cycles													
		5½ gallons of water per sack of cement									6½ gallons of water per sack of cement				
Source	Type	10 cycles	20 cycles	30 cycles	40 cycles	50 cycles	60 cycles	70 cycles	80 cycles	100 cycles	10 cycles	20 cycles	30 cycles	40 cycles	50 cycles
A	I	94	101	102	104	100	101	101	101	99	99	100	100	98	96
A	IS	99	104	102	107	102	104	105	106	106	101	102	103	103	101
B	I	97	104	102	103	99	103	102	99	100	100	100	100	98	96
B	IS	99	105	103	106	103	106	107	106	106	101	104	104	104	103
C	I	99	106	103	102	96	98	98	97	94	100	101	101	100	96
C	IS	101	105	102	105	103	103	104	103	102	100	102	103	102	101
D	I	100	104	104	103	102	103	101	101	98	101	103	103	100	96
D	IS	101	104	102	103	103	105	104	104	103	98	98	98	99	100
E	I	102	104	104	102	101	101	101	101	102	99	100	99	98	96
E	IS	98	102	102	102	101	104	104	105	105	101	102	103	102	100

[1] Specimens were 3- by 4- by 16-inch beams frozen and thawed in accordance with ASTM Method C–292 for slow freezing and thawing in water. Each value is an average of tests on 5 beams.

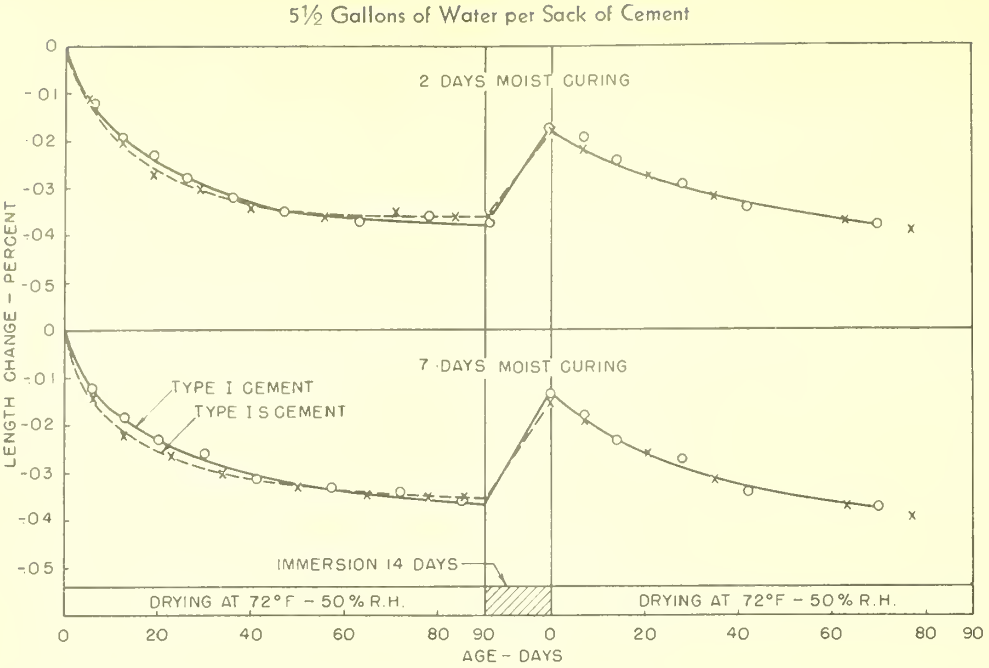

Figure 3.—Comparison of drying shrinkage of portland cement concrete (type I) and portland blast-furnace slag cement concrete (type IS) from 5 sources. (Water-cement ratio: 5½ gallons of water per sack of cement.)

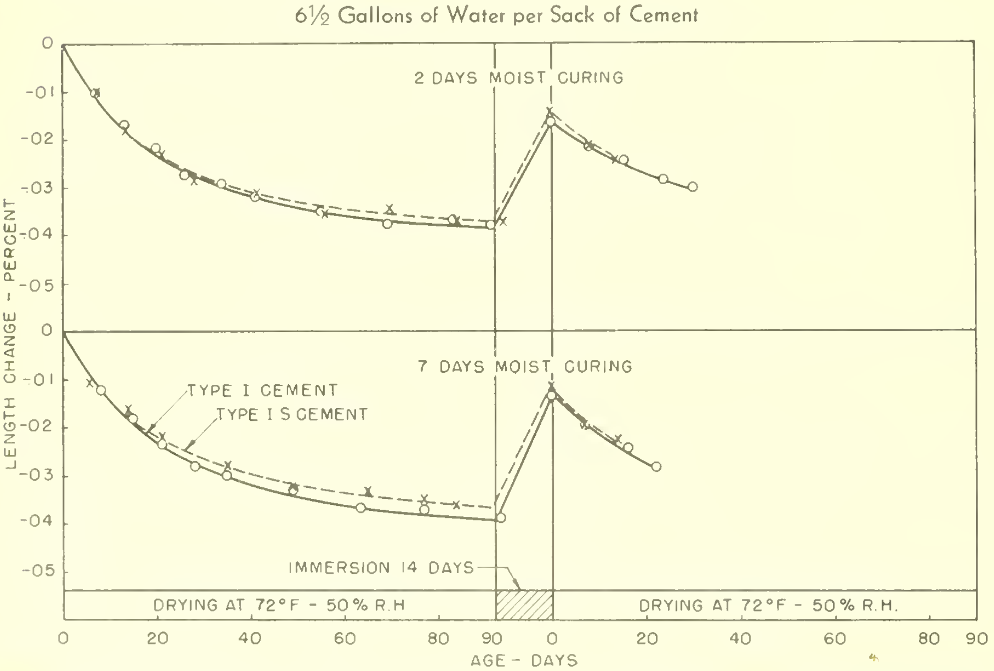

Figure 4.—Comparison of drying shrinkage of portland cement concrete (type I) and portland blast-furnace slag cement concrete (type IS) from 5 sources. (Water-cement ratio: 6½ gallons of water per sack of cement.)

ages of 3 and 7 days, the concretes prepared with portland cement have a higher average modulus than those made with slag cement. At an age of 28 days, the portland cement concretes have a slightly higher average modulus, but at greater ages the reverse is found. All concretes prepared with portland cement from source E, with one exception, have higher moduli than their equivalents prepared with slag cement. However, the moduli for the concretes prepared with slag cement from source E increased with age at a more rapid rate than those for concretes prepared with portland cement from the same source. It is expected that at an age of one year, the moduli for the slag cement concrete will be higher than those for portland cement concrete.

Static Modulus of Elasticity

The static modulus of elasticity was determined on a number of 6- by 12-inch cylinders that were molded from the same batches of concrete used to prepare beams on which the sonic modulus was determined. These concretes containing 7½ gallons of water per sack of cement were tested at an age of 90 days. The results are given in table 9.

It is interesting to note that the difference between the sonic and static moduli for the slag cement concretes is about one-half of that for the portland cement concretes. More extensive data for the static modulus of elasticity will be included in the final report.

Freezing and Thawing Tests

Freezing and thawing tests on concretes prepared with 5½ and 6½ gallons of water per sack of cement are being continued, and the results obtained thus far are shown in table 10. Although the values for N^2 show some slight differences between concretes prepared with the two types of cement, it would probably be desirable to defer consideration of these results until more significant values have been obtained.

Shrinkage Tests

Figures 3 and 4 show the average results of shrinkage tests for concrete prepared with the two types of cement and 5½ or 6½ gallons of water per sack of cement. Some of the specimens tested were moist cured for 2 days; others were moist cured for 7 days prior to the beginning of measurements for shrinkage. The specimens were stored in room air at 72° F. and 50 percent relative humidity for 90 days, and length measurements were made at frequent intervals. The specimens were then immersed in water for 14 days and a second determination of shrinkage made. The differences found to date between concrete specimens prepared with the two types of cement were too small to be significant. It appears that no more concern need be expressed over the shrinkage of concrete prepared with slag cement than over concrete prepared with portland cement, provided the amount of water used is kept within reasonable limits.

Scaling Tests with Calcium Chloride

Concrete slabs prepared with each of the cements for freezing and ice removal tests have been exposed to the atmosphere and subjected to five cycles of freezing and thawing. Although some scaling has already been noted, no qualitative estimates of the effect of the different cements on this scaling can be made.

Relation Between Gross Weights of Motor Trucks and Their Horsepower

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH
BUREAU OF PUBLIC ROADS

Reported by CARL C. SAAL, Chief,
Vehicle Operations Branch

This article discusses the results of a study made in 1950 of the relation between the gross weight and engine horsepower of commercial vehicles. Over 10,000 commercial vehicles, ranging in type from 2-axle trucks to 6- and 7-axle truck combinations, were sampled from the everday traffic in 39 States.

The primary purpose of the study was to develop average values of gross weight-horsepower ratios that could be used in conjunction with the results of studies of travel time, grade-climbing ability, and accelerating ability to estimate time savings to truck operators through more favorable location of highways. A secondary but important purpose was the determination of the impact of proposed minimum performance requirements of gross weight and horsepower on the trucking industry.

Previous studies of hill-climbing ability of commercial vehicles have indicated the futility of attempting to legislate and enforce performance requirements that would materially alleviate congestion caused by the slow-moving vehicles on hills. A minimum requirement in terms of weight and horsepower might be desirable, however, from the standpoint of safer vehicle operation; it is believed that the adoption of such a requirement would result in a better balance between the weight of the vehicle and the capabilities of its brakes, tires, and other components. A minimum requirement would also provide the highway engineer with a fixed base for establishing highway design standards.

For a ratio of 400 pounds gross weight per horsepower, a value commonly accepted as tolerable within the automotive industry and the one used for calculating performance for highway design purposes, the percentages of commercial vehicles in 1950 having weight-power ratios greater than 400:1 were as follows: 3-axle trucks, 10 percent; 2-axle truck-tractors with 1-axle semitrailers, 13 percent; 2-axle truck-tractors with 2-axle semitrailers, 41 percent; and all other combinations, 57 percent.

Comparisons of weight-power ratios by geographical regions and vehicle types show definitely that the ratios increase with the number of axles. It is also evident that weight-power ratios increase substantially with an increase in gross weight, irrespective of vehicle type.

Weight-power ratios, developed from 1955 data collected in a study of braking ability of commercial vehicles, show an overall improvement in performance of at least 10 percent during the period 1950–55.

IN 1950, each State highway department was asked to obtain information in conjunction with the annual loadometer survey that could be used to determine the ratio between gross vehicle weight and engine horsepower for commercial vehicles operating in the everyday traffic. In addition to the data usually recorded, the vehicle make, model, and year model were obtained for trucks and truck combinations at weighing stations located on routes having a heavy concentration of truck traffic.

Data were obtained from all except nine States (Florida, Idaho, Illinois, Kansas, Maryland, Michigan, Pennsylvania, Tennessee, and Wisconsin) and the District of Columbia. This survey of vehicles, selected at random, yielded a total sample of 10,726 vehicles of various types. The data collected in the annual loadometer surveys were supplemented by similar information obtained from brake tests on 782 vehicles in 1949 and 862 vehicles in 1955. The brake test data were used in the present study to investigate the trend in weight-power ratios.

Terminology

Four terms used in this article should be clearly understood. Definitions of the terms follow:

Gross horsepower.—The brake horsepower of the engine, operating without such accessories as fan, air compressor, generator, and muffler, that is available at the clutch or its equivalent.

Net horsepower.—The brake horsepower of the engine, operating with all its normal accessories, that is available at the clutch or its equivalent. It is the gross horsepower minus the horsepower absorbed by such accessories as fan compressor, muffler, and generator.

Weight-power ratio.—The ratio of the gross weight of the vehicle or combination of vehicles to net horsepower of the powered unit. For example, if the gross weight of a vehicle is 60,000 pounds and the net horsepower is 150, the ratio is 400 pounds per horsepower.

Rate of rise and fall.—The total rise and fall for any section of highway divided by the length of section in hundreds of feet. If the total rise and fall is 640 feet and the length of section is 20,000 feet, then the rate of rise and fall is 3.2 feet per 100 feet. It is not to be confused with the percent of grade. It is equivalent to the average percent of grade only when either the rise or fall is 100 percent of the total rise and fall.

Purpose of Investigation

The principal objective of the survey was to provide frequency distributions and average values of weight-power ratios that could

Table 1.—Cumulative frequency distribution of weight-horsepower ratios of 2-axle, 6-tire trucks weighed in the 1950 loadometer study

Region	Number of vehicles	Percentage of 2-axle, 6-tire trucks with weight-horsepower ratios less than—							Average ratio
		100	150	200	250	300	350	400	
New England	795	33.8	68.4	84.0	95.3	99.1	100.0	----	135
Middle Atlantic	100	22.0	59.0	73.0	91.0	97.0	100.0	----	154
South Atlantic (North)	170	40.6	62.9	80.0	92.9	99.4	100.0	----	139
South Atlantic (South)	336	47.3	66.3	82.1	93.4	99.1	100.0	----	131
East North Central	74	62.1	87.8	93.2	98.6	100.0	----	----	106
East South Central	330	45.8	70.0	82.7	94.8	99.1	99.7	100.0	130
West North Central	538	25.6	53.8	75.4	93.2	99.2	100.0	----	152
West South Central	769	33.7	73.5	90.2	97.7	99.4	100.0	----	129
Mountain	206	24.8	53.9	74.8	88.4	96.6	100.0	----	156
Pacific	324	35.2	70.4	82.7	96.3	99.7	100.0	----	133
United States total	3,642	35.1	66.5	82.8	94.8	99.0	99.9	100.0	135

Table 2.—Cumulative frequency distribution of weight-horsepower ratios of 2-axle truck-tractors with 1-axle semitrailers weighed in the 1950 loadometer study

Region	Number of vehicles	Percentage of 2-axle truck-tractors with 1-axle semitrailers having weight-horsepower ratios less than—											Average ratio
		100	150	200	250	300	350	400	450	500	550	600	
New England	900	------	5.3	22.5	35.7	55.8	77.7	90.8	97.9	99.3	99.9	100.0	281
Middle Atlantic	290	0.3	2.4	15.2	27.6	39.0	56.9	76.6	90.7	97.6	98.3	98.6	324
South Atlantic (North)	217	1.9	8.0	20.1	33.1	50.3	74.0	90.3	98.2	99.1	100.0	------	288
South Atlantic (South)	467	.2	4.5	22.7	34.0	52.0	80.0	93.1	97.4	100.0	------	------	281
East North Central	118	------	5.1	17.8	30.5	46.6	72.0	89.8	97.4	99.1	100.0	------	295
East South Central	306	1.0	6.9	30.4	39.9	55.9	76.5	91.8	97.7	99.3	100.0	------	274
West North Central	755	------	2.2	11.9	21.7	34.0	57.3	82.5	92.7	98.0	99.6	99.7	325
West South Central	601	1.7	6.7	29.7	44.8	53.8	70.6	86.1	95.9	99.4	99.9	99.9	281
Mountain	198	.5	6.6	13.2	27.3	48.5	70.7	85.4	93.0	98.5	99.5	100.0	302
Pacific	48	10.0	26.0	70.0	74.0	82.0	90.0	92.0	98.0	98.0	100.0	------	208
United States total	3,900	.7	5.3	21.6	33.8	49.1	70.8	87.6	95.8	99.0	99.7	99.8	294

Table 3.—Cumulative frequency distribution of weight-horsepower ratios of truck combination units with 4 or more axles weighed in the 1950 loadometer study

Region	Number of vehicles	Percentage of combination units with 4 or more axles having weight-horsepower ratios less than—										Average ratio
		150	200	250	300	350	400	450	500	550	600	
New England	110	0.9	12.7	20.9	33.6	50.0	68.2	79.1	90.9	93.6	95.4	352
Middle Atlantic	131	------	5.3	16.8	27.5	37.4	57.3	81.7	93.9	97.7	98.5	367
South Atlantic (North)	168	1.2	14.3	19.7	32.8	52.4	72.0	86.9	95.8	97.0	97.6	342
South Atlantic (South)	85	2.4	17.7	31.8	47.1	65.9	75.3	87.1	93.0	97.7	97.7	316
East North Central	97	3.1	14.4	25.7	29.8	35.0	43.3	55.7	69.1	77.3	87.6	416
East South Central	79	5.0	27.8	36.7	44.3	54.4	69.6	82.3	92.4	94.9	100.0	323
West North Central	916	.2	6.3	16.4	25.4	36.5	52.4	67.7	82.3	91.9	96.2	386
West South Central	347	2.3	13.0	25.1	30.9	40.1	52.8	73.8	88.8	95.7	97.4	344
Mountain	597	.8	8.9	21.1	31.8	39.5	53.5	65.3	79.2	90.9	97.1	381
Pacific	152	1.3	8.5	33.5	37.4	42.7	54.6	69.1	78.3	94.1	98.0	364
United States total	2,682	1.1	9.9	21.4	30.5	41.0	55.8	71.0	84.2	92.8	96.6	371

be correlated with the results of other studies of travel time, grade-climbing ability, and accelerating ability of trucks. For example, the results of this study may be used in conjunction with the relations established between travel time and weight-horsepower ratios in a previous article[1] in order to estimate the savings in time that accrue to truck operators through more favorable location of a highway.

Table 4.—Summary of average weight-horsepower ratios and gross weights for single-unit trucks

Region	2-axle, 4-tire			2-axle, 6-tire			3-axle		
	Number of vehicles	Average ratio	Average weight	Number of vehicles	Average ratio	Average weight	Number of vehicles	Average ratio	Average weight
New England	119	73	5,700	795	135	12,500	39	227	21,800
Middle Atlantic	-----	-----	-----	100	154	15,300	21	290	27,900
South Atlantic (North)	5	67	5,600	170	139	12,700	53	262	25,500
South Atlantic (South)	3	87	7,300	336	131	11,900	3	209	17,000
East North Central	2	47	5,800	74	106	9,300	6	117	11,000
East South Central	6	78	6,900	330	130	11,500	51	185	16,800
West North Central	8	108	8,600	538	152	12,100	28	332	30,300
West South Central	52	73	6,100	769	129	11,400	13	240	25,100
Mountain	9	67	5,800	206	156	13,800	19	271	32,100
Pacific	35	76	6,100	324	133	11,800	30	232	26,400
United States total	239	75	6,000	3,642	135	12,100	263	244	24,100

Table 5.—Summary of average weight-horsepower ratios and gross weights for truck-tractors with semitrailers

Region	2-axle truck-tractor with 1-axle semitrailer			2-axle truck-tractor with 2-axle semitrailer			3-axle truck-tractor with 2-axle semitrailer		
	Number of vehicles	Average ratio	Average weight	Number of vehicles	Average ratio	Average weight	Number of vehicles	Average ratio	Average weight
New England	900	281	36,500	109	347	48,700	1	865	77,400
Middle Atlantic	290	324	40,000	131	367	49,300	-----	-----	------
South Atlantic (North)	217	288	33,000	165	342	45,100	2	264	48,000
South Atlantic (South)	467	281	31,100	85	316	39,300	-----	-----	------
East North Central	118	295	29,500	55	367	39,600	2	585	63,000
East South Central	306	274	28,300	78	322	39,200	1	357	40,000
West North Central	755	325	32,400	773	385	46,100	141	395	51,200
West South Central	601	281	27,400	339	341	42,100	8	484	67,900
Mountain	198	302	31,600	215	327	43,000	290	421	58,800
Pacific	48	208	21,000	41	344	41,100	38	372	53,700
United States total	3,900	294	31,600	1,991	357	44,500	483	411	56,300

A secondary but important purpose was to provide means for determining the impact of proposed minimum performance requirements on the trucking industry. In this phase, the results may be used to find the percentage of vehicles that would be affected by the 450-pound gross weight per net horsepower performance requirement recently enacted in Pennsylvania. This may also be done in the case of a requirement expressed in x miles per hour on a y percent grade, since hill-climbing ability is proportional to the ratio. A ratio of 450 is roughly equivalent to 20 miles per hour on a 2½-percent grade.

If performance requirements are to be enacted, the use of a weight-power ratio instead of a grade-climbing ability expression is recommended. A weight-power ratio does not impose limitations on vehicle design and operation and may be practically enforced by weighing the vehicle.

A performance requirement should not be predicated on the idea that it would eliminate the congestion that results from slow-moving vehicles on hills. The results of hill-climbing studies[2] conducted by the Bureau of Public Roads and the Arizona Highway Department have shown the futility of trying to legislate and enforce a requirement that would materially alleviate congestion. It is believed that the eventual solution must be preceded by the recognition of the following conditions: (1) The operating characteristics of freight

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[1] *Time and gasoline consumption in motor truck operation as affected by the weight and power of vehicles and the rise and fall in highways*, by Carl Saal. Highway Research Board Research Report No. 9-A, February 1950.

[2] *Hill-climbing ability of motor trucks*, by Carl C. Saal. PUBLIC ROADS, vol. 23, No. 3, May 1942. *Survey of uphill speeds of trucks on mountain grades*, by Wm. E. Willey, Proceedings of the Twenty-Ninth Annual Meeting of the Highway Research Board, 1949, pp. 304–310; also *Truck congestion on uphill grades*, Highway Research Board Bulletin 104, 1955, pp. 21–33.

Table 6.—Summary of average weight-horsepower ratios and gross weights for trucks with trailers and truck-tractors with semitrailers and trailers

Region	2-axle truck with 2-axle trailer			5-axle combinations of trucks with trailers and truck-tractor-semitrailers and trailers			6- and 7-axle combinations of trucks with trailers and truck-tractor-semitrailers and trailers		
	Number of vehicles	Average ratio	Average weight	Number of vehicles	Average ratio	Average weight	Number of vehicles	Average ratio	Average weight
South Atlantic (North)	-----	-----	-----	-----	-- --	--- -	1	467	56,000
East North Central	11	532	60,100	27	459	53,400	2	368	62,500
West North Central	1	340	62,300	-----	-----	-----	1	384	70,300
Mountain	7	373	42,800	61	365	51,700	24	430	62,300
Pacific	1	422	43,500	48	366	50,500	24	384	53,100
United States total	20	461	53,300	136	384	51,600	52	406	58,100

Table 7.—Summary of average weight-horsepower ratios and gross weights for 2-axle trucks with 2-axle trailers and combination vehicles with 5 or more axles

Region	Number of vehicles	Average ratio	Average weight
New England	1	865	77,400
South Atlantic (North)	3	332	50,700
East North Central	42	480	56,100
East South Central	1	357	40,000
West North Central	143	394	51,400
West South Central	8	484	67,900
Mountain	382	412	57,600
Pacific	111	373	52,100
United States total	691	407	55,400

Table 8.—Variation of weight-horsepower ratios and gross weights for all commercial vehicle types

Region	Number of vehicles	Average weight-power ratios for gross weights (in 1,000 pounds) ranging from—									Average weight-power ratios
		3.0 to 9.9	10.0 to 19.9	20.0 to 29.9	30.0 to 39.9	40.0 to 49.9	50.0 to 59.9	60.0 to 69.9	70.0 to 79.9	80.0 to 89.9	
New England	1,963	86	155	216	275	336	387	434	574	--------	212
Middle Atlantic	542	93	160	214	306	370	404	446	474	--------	302
South Atlantic (North)	613	88	158	231	312	354	431	436	--------	460	257
South Atlantic (South)	894	84	166	242	320	357	467	--------	--------	--------	227
East North Central	297	85	163	252	338	424	488	566	643	--------	282
East South Central	772	95	162	252	342	372	434	454	--------	--------	210
West North Central	2,245	92	172	250	342	390	455	495	538	597	308
West South Central	1,782	91	162	243	356	400	461	498	585	634	221
Mountain	1,029	88	163	240	284	363	408	471	521	573	316
Pacific	589	88	158	218	280	371	439	471	508	--------	201
United States total	10,726	89	162	237	320	368	433	475	532	568	253

and passenger vehicles are definitely dissimilar; (2) the two classes cannot be designed to approach similarity in the foreseeable future without grave injustice to one or both; (3) the public interest requires that highways provide adequately for both classes of vehicles; and (4) by appropriate highway design, both can be provided for without undue interference in their movement over the highways.

A minimum performance requirement in terms of weight and power is believed to be desirable. However, this position is based primarily on the belief that the adoption of such a requirement would result in a better balance between the weight of a vehicle and the capabilities of its brakes, tires, and other components. An improved balance between design capacities of such components and gross vehicles weights should result in safer vehicle operation. It would also give the highway engineer a fixed base for establishing highway design standards to accommodate both freight and passenger vehicles efficiently.

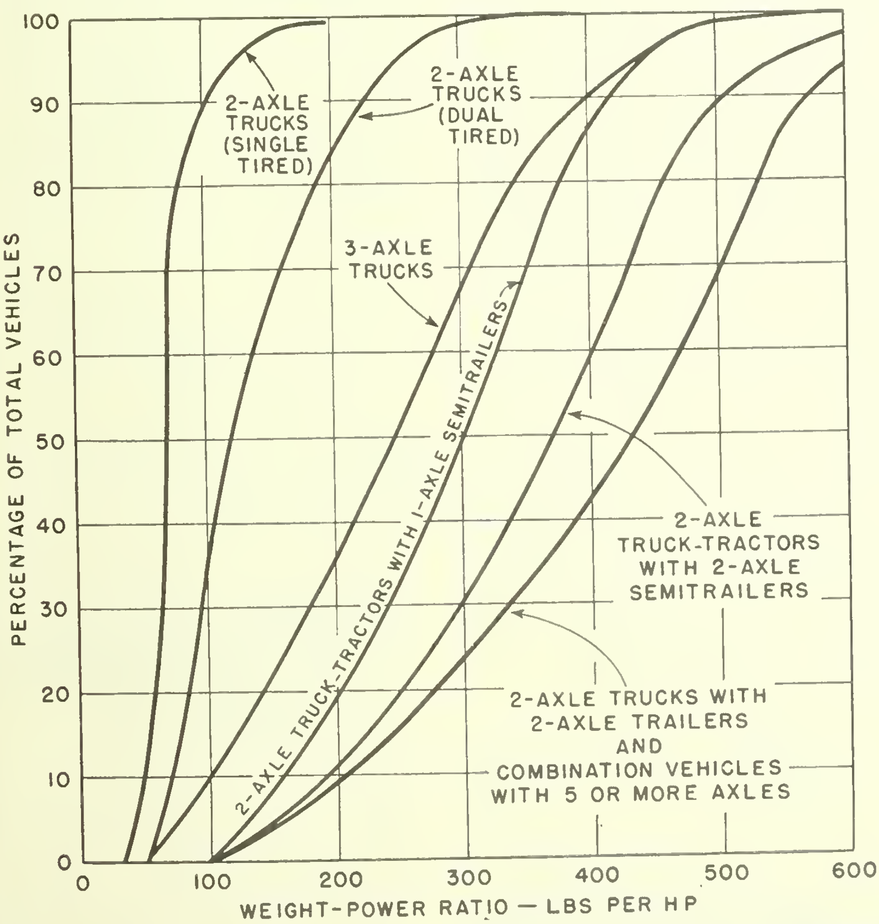

Figure 1.—Cumulative frequency distributions of weight-power ratios for commercial vehicles weighed in the 1950 loadometer study.

Vehicle Type Code

In some instances it is convenient to describe vehicle types by the codes generally used. Each digit indicates the number of axles of a vehicle or of a unit of a vehicle combination. A single digit, or the first digit of a group symbol, represents a single-unit truck, and if followed by an "S," it represents a truck-tractor. The "S" designation, of course, represents a semitrailer. A digit without an "S," in the second or third position in a group symbol, represents a full trailer.

2 =2-axle single-unit truck
3 =3-axle single-unit truck
2–S1 =2-axle truck-tractor with 1-axle semitrailer
2–S2 =2-axle truck-tractor with 2-axle semitrailer
3–S2 =3-axle truck-tractor with 2-axle semitrailer
2–2 =2-axle truck with 2-axle trailer
2–3 =2-axle truck with 3-axle trailer
3–2 =3-axle truck with 2-axle trailer
3–3 =3-axle truck with 3-axle trailer
2–S1–2 =2-axle truck-tractor with 1-axle semitrailer and 2-axle trailer
2–S2–2 =2 axle truck-tractor with 2-axle semitrailer and 2-axle trailer
2–S2–3 =2-axle truck-tractor with 2-axle semitrailer and 3-axle trailer
3–S1–2 =3-axle truck-tractor with 1-axle semitrailer and 2-axle trailer

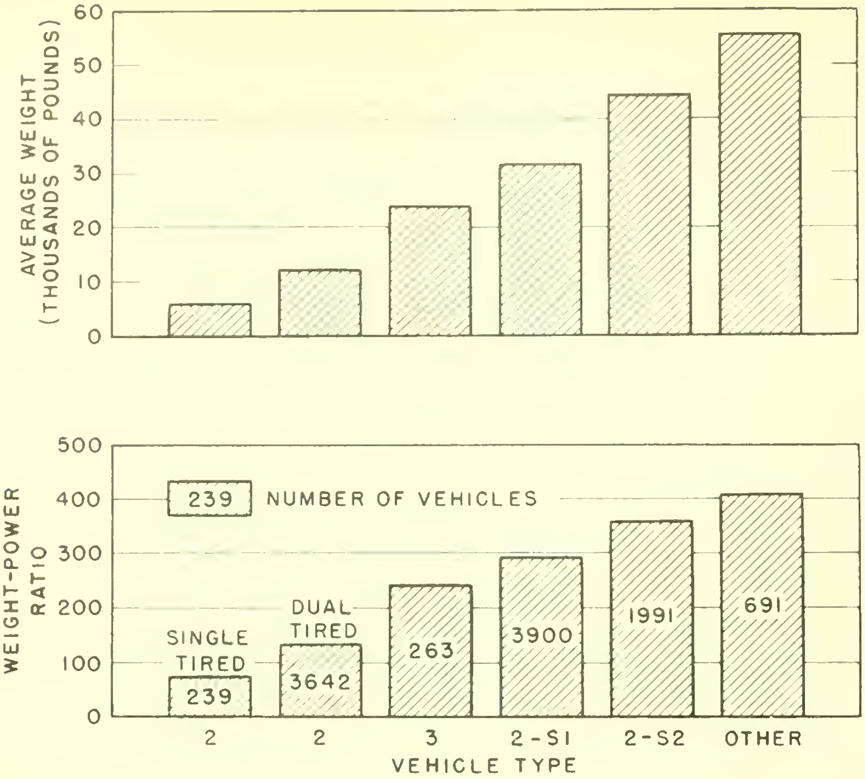

Figure 2.—Average weight-power ratios and gross weights for commercial vehicles weighed in the 1950 loadometer study.

Procedure Followed

The first step in the analysis was the determination of the net horsepower for each of the vehicles sampled. This was accomplished by using vehicle specifications, furnished by individual manufacturers and the Automobile Manufacturers Association, in conjunction with the make and model recorded in the field. In those instances, where it was possible to find only the gross horsepower, the net horsepower was assumed to be 90 percent of that value. The weight-power ratio was then calculated by dividing the gross weight by the net horsepower.

The next step was to group the sample data according to vehicle type. This step was followed by further subdividing the sample for each vehicle type by geographical regions. These regions, omitting the States not reporting, were composed as follows:

New England.—Connecticut, Maine, Massachusetts, New Hampshire, Rhode Island, and Vermont. *Middle Atlantic.*—New Jersey and New York. *South Atlantic (North).*—Delaware, Virginia, and West Virginia. *South Atlantic (South).*—Georgia, North Carolina, and South Carolina. *East North Central.*—Indiana and Ohio. *East South Central.*—Alabama, Kentucky, and Mississippi. *West North Central.*—Iowa, Minnesota, Missouri, Nebraska, North Dakota, and South Dakota. *West South Central.*—Arkansas, Louisiana, Oklahoma, and Texas. *Mountain.*—Arizona, Colorado, Montana, Nevada, New Mexico, Utah, and Wyoming. *Pacific.*—California, Oregon, and Washington.

When the sample had been sorted by vehicle type and region, three types of analyses were made. Frequency distributions of weight-power ratios grouped in class intervals of 50 pounds gross weight per horsepower were first made for each vehicle type by region and total sample. Then the average weight and ratio were determined for each vehicle type by region and total sample. A third analysis was made to determine the relation existing between the ratio and gross weight. This was accomplished by distributing the vehicles by type and geographical region into class intervals of 10,000 pounds gross weight and by calculating an average ratio for each class interval.

Results of Survey

The cumulative frequency distributions of the weight-power ratios by regions are shown in table 1 (p. 233) for the 2-axle, 6-tire trucks; in table 2 for the 2-axle truck-tractors with 1-axle semitrailers; and in table 3 for combination vehicles with 4 or more axles. Also shown in this series of tables are the average ratios and the number of vehicles in the sample. The 2-axle, 4-tire trucks and the 3-axle trucks are not considered because of the small sample in most regions.

A summary of the cumulative frequency distributions for the several types of vehicles is shown in figure 1. For a ratio of 400 pounds gross weight per horsepower, a value commonly accepted as tolerable within the automotive industry and the one used for calculating performance for highway design purposes, the cumulative percentages of vehicles having ratios less than 400 pounds per horsepower were as follows: 100 percent for both 2-axle, 4- and 6-tire trucks, 90 percent for 3-axle trucks, 87 percent for 2-axle truck-tractors with 1-axle semitrailers, 59 percent for 2-axle truck-tractors with 2-axle semitrailers, and 43 percent for all the other combination vehicles. The comparison in figure 1 indicates that the hill-climbing and accelerating ability of commercial vehicles vary considerably for the different vehicle types. This dissimilarity in power characteristics is analogous with braking ability [3] for the various types of commercial vehicles selected from the everyday traffic.

Tables 4–7 give the average ratios and weights for each vehicle type by geographical regions. National totals by vehicle types are

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[3] *Braking performance of motor vehicles*, by Carl C. Saal and F. William Petring. Published by the Bureau of Public Roads, 1954.

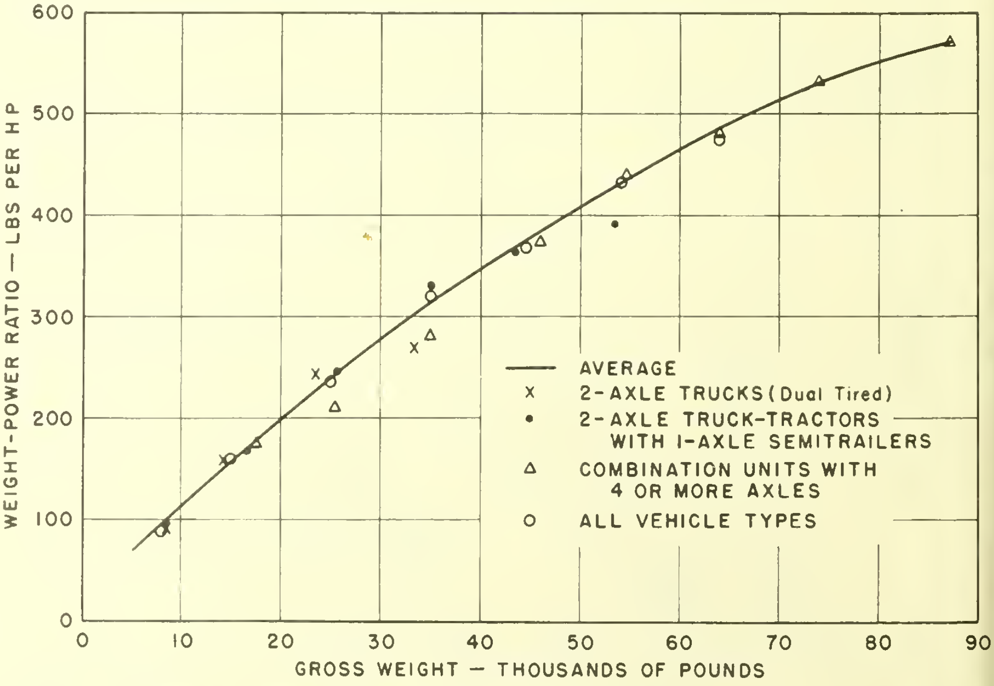

Figure 3.—Relation of weight-power ratios and gross weights for commercial vehicles weighed in the 1950 loadometer study.

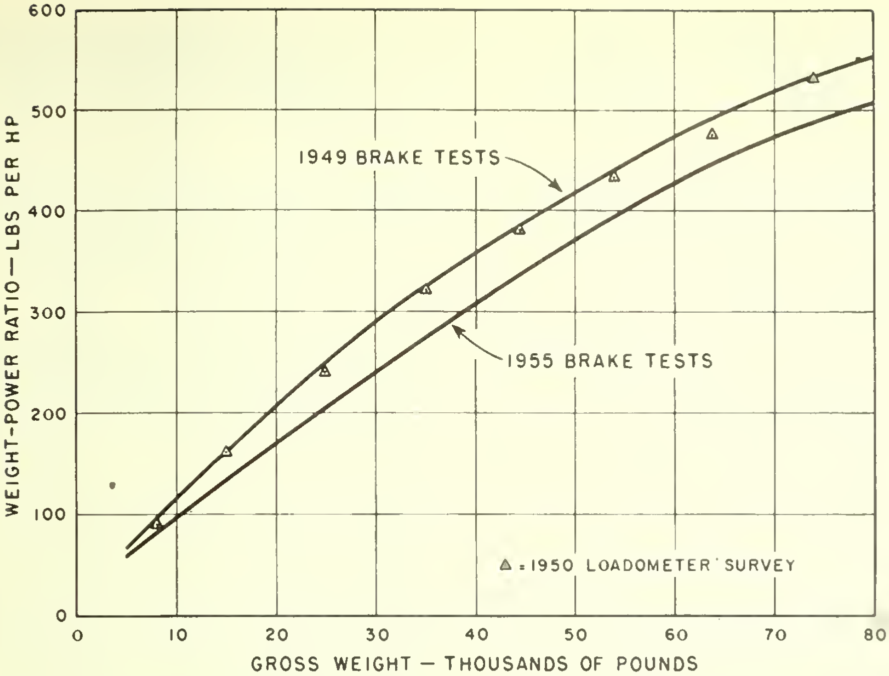

Figure 4.—Trends in weight-power ratios from 1949 to 1955, based on average data for all commercial vehicle types.

compared in figure 2. This presentation of the data emphasizes again the wide variation in performance that may be expected of the various vehicle types. There is a definite increase in weight-power ratios with an increase in the number of axles.

The data in table 8 show that the ratios increase with an increase in gross weight. It is further indicated in figure 3 that the same relation exists, irrespective of vehicle type.

Trend in Ratios

The levels of performance indicated by the ratios presented thus far in this article are on the conservative side for two reasons. It is rather common for a commercial truck manufacturer to have several optional engine sizes for a particular chassis model. The model of the engine installed was sometimes difficult for the weighing party to determine, and where there was any doubt, the net horsepower for the smallest engine size was used in computing the ratio. A second reason for the conservative evaluation is the age of the data which were collected over 5 years ago. During this period there has been a decided upward trend in net engine horsepower which has been offset to some extent by an increase in gross vehicle weights.

In studies conducted by the Arizona Highway Department,[4] it was reported that the crawl speed of heavily loaded over-the-road combinations climbing a 6-percent grade increased from 7 miles per hour in 1948 to 12.5 in 1952, an improvement of about 78 percent. However, the data from the brake tests shown in figure 4 indicate that the ratios have probably not decreased in this proportion for the country as a whole, although there has been an appreciable improvement.

The curves in figure 4, which are similar to the one presented in figure 3, were plotted from data recorded at a location in Maryland, Michigan, and California. The points in figure 3, which represent the average ratios for all vehicles sampled in 1950, are also plotted in figure 4 (triangular symbols). It is apparent that the average ratios derived from the 1950 loadometer study follow closely the curve for the 1949 brake test data.

The improvement in ratios indicated by figure 4 amounts to about 15 percent in the average ratio for gross weights ranging from 10,000 to 40,000 pounds. Above that weight, the improvement decreases to about 8 percent for a gross weight of 80,000 pounds. It may be stated that the improvement, which represents a decrease in the ratio, averages at least 10 percent.

Table 9 compares the 1950 average ratios for the various vehicle types with those obtained from the 1949 and 1955 brake test samples of vehicles. In general, there is close agreement between the 1949 and 1950 values as was the case in figure 4. This is indicated by the rather small variation, except in a few instances, between the percentage improvement from 1949 to 1955 and from 1950 to 1955. The exceptions occur where the samples were very small. The overall improvement between 1949 and 1955 was about 12 percent, which checks closely with that estimated from the curves plotted in figure 4.

Another check on the trend in weight-power ratios was made by comparing the 1950 and 1955 percentile values obtained from cumulative frequency distributions of the ratios for the various types of vehicles. A comparison of 15-, 50-, and 85-percentile values read from figure 1 for 1950 and figure 5 for 1955 indicated that the overall improvement was negative in the case of 2-axle, 6-tire trucks; very slight for 3-axle trucks, 2-axle trucks with

Table 9.—Comparison of average weight-power ratios by vehicle types for 1949, 1950, and 1955

Vehicle type [1]	Number of vehicles			Average weight-power ratios			Percentage improvement in weight-power ratios	
	1949	1950	1955	1949	1950	1955	1949–55	1950–55
Single-unit trucks:								
2-axle, 4-tire	19	239	99	81	75	57	30	24
2-axle, 6-tire	275	3,642	272	142	135	142	-------	−5
3-axle	38	263	67	227	244	231	−2	5
Truck-tractors with semitrailers:								
2–S1	228	3,900	117	291	294	264	9	10
2–S2	87	1,991	145	369	357	301	18	16
3–S2	46	483	57	422	411	348	18	15
Other truck combinations:								
2–2	8	20	17	405	461	331	18	28
2–3, 3–2, and 2–S1–2	51	136	71	394	384	418	−6	−9
3–3, 2–S2–2, and 3–S2–2	30	52	17	434	406	416	4	−2
Total vehicles	782	10,726	862	----	----	----	-------	-------
Weighted averages	----	------	----	260	253	228	12	10

[1] For an explanation of code, see p. 235.

Table 10.—Example for determining daily time savings by using weight-power ratios calculated for different gross vehicle weight groups

Gross vehicle weight		Number of vehicles	Weight-power ratio [1]	Composite travel time [2]			Time saved per vehicle	Total time saved
Class interval	Average			Route A	Route B	Savings		
1,000 lbs.	*Pounds*			*Min./mile*	*Min./mile*	*Min./mile*	*Minutes*	*Hours*
5–14.9	9,800	545	100	1.40	1.25	0.15	15.0	136
15–24.9	19,600	485	170	1.75	1.40	.35	35.0	284
25–34.9	29,300	176	240	2.10	1.60	.50	50.0	146
35–44.9	41,000	111	310	2.40	1.75	.65	65.0	120
45–54.9	48,300	91	350	2.65	1.85	.80	80.0	121
55 and over	59,900	32	410	2.95	2.10	.85	85.0	45
Total	------	1,440	------	------	------	------	------	852

[1] Weight-power ratios from fig. 3 multiplied by 0.88.
[2] Rate of rise and fall: Route A, 3.7; Route B, 2.3. Each route was 100 miles long.

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[4] See footnote 2, p. 234.

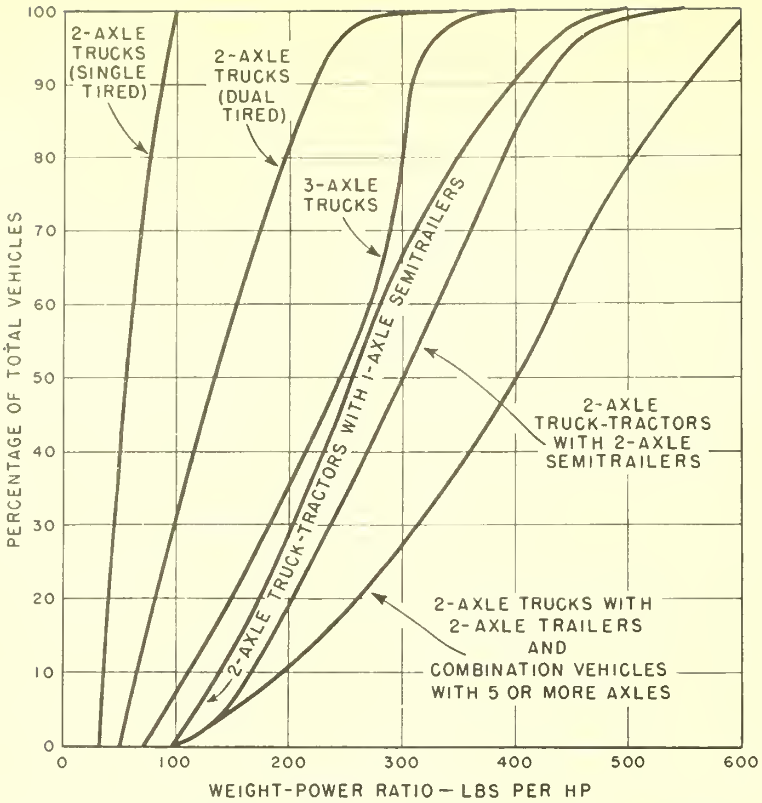

Figure 5.—Cumulative frequency distributions of weight-power ratios for commercial vehicles weighed in the 1955 brake test study.

Table 11.—Example for determining daily time savings by using average values of weight-power ratios for the different vehicle types

Vehicle type	Number of vehicles	Weight-power ratio [1]	Composite travel time [2]			Time saved per vehicle	Total time saved
			Route A	Route B	Savings		
Single unit trucks:			*Min./mile*	*Min./mile*	*Min./mile*	*Minutes*	*Hours*
2-axle, 6-tire	694	120	1.50	1.35	0.15	15.0	173
3-axle	70	215	2.00	1.55	.45	45.0	52
Truck combinations:							
2-axle truck-tractor with 1-axle semi-trailer	338	260	2.25	1.70	.55	55.0	309
2-axle truck-tractor with 2-axle semi-trailer	338	315	2.45	1.80	.65	65.0	366
Total	1,440	------	------	------	------	------	900

[1] Average weight-power ratios from tables 4 and 5 multiplied by 0.88.
[2] Rate of rise and fall: Route A, 3.7; Route B, 2.3. Each route was 100 miles long.

Table 12.—Percentage of vehicles weighed in the 1955 brake tests that could not meet indicated performance levels

Vehicle type	Percentage of vehicles with weight-power ratios greater than—					
	250:1	300:1	350:1	400:1	450:1	500:1
Single-unit trucks:						
2-axle, 6-tire	3	1	--------	--------	--------	--------
3-axle	48	21	2	--------	--------	--------
Truck combinations:						
2-axle truck-tractor with 1-axle semi-trailer	53	34	20	10	2	--------
2-axle truck-tractor with 2-axle semi-trailer	66	50	34	17	5	1
All other combinations	82	73	62	50	35	22

2-axle trailers grouped with combinations of 5 or more axles; and appreciable for 2-axle, 4-tire trucks and 2-axle truck-tractors with 1- and 2-axle semitrailers. The greatest improvement among the combination vehicles was observed for 2-axle truck-tractors with 2-axle semitrailers.

Time Savings Computed from Weight-Power Ratios

If the normal distribution of vehicle weights is known for a given route, the time savings that might accrue to commercial vehicles through the improvement of the existing route or the construction of a route on another location may be computed by using ratios selected from the curve in figure 3. An example of the computations involved is contained in table 10 for two routes, each 100 miles in length, but with different rates of rise and fall, 3.7 against 2.3. The weight-power ratios were taken from the curve in figure 3 for the average weight of the group and then corrected to present conditions by multiplying by the factor 0.88. The midpoint of the group may be used instead of the average gross weight. The travel time in minutes per mile for Routes A and B was then read from the chart on page 54 of the Highway Research Board Research Report No. 9–A [6] for the given ratio and the respective rate of rise and fall. The total time saved was obtained by first multiplying the difference between the travel times for Routes A and B by the length in miles to compute the savings per vehicle, and then multiplying that value by the daily number of vehicles in the particular weight group.

When the distribution of commercial vehicles by weight is not known, the travel time may be estimated in a similar manner by using the average values of the ratios contained in tables 4–7 and the number of vehicles by type obtained from a visual traffic classification survey. The example in table 11 shows how this can be accomplished. The classification of vehicles by type is for the same sample of vehicles used to demonstrate the previous method.

Impact of Performance Requirements

The percentage of the vehicles sampled in 1955 that could not meet various levels of performance is shown in table 12. The percentages were determined from figure 5. It appears from this tabulation that a value of 400 pounds gross weight per horsepower is tolerable for highway design purposes. It is also evident that a performance requirement of less than a 400:1 ratio would affect a sizable number of combination units, especially those with more than 4 axles.

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[6] See footnote 1, p. 234.

The Efficiency of Public Transit Operation in the Utilization of City Streets

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by WILLIAM P. WALKER, Highway Transport Research Engineer, and ROY A. FLYNT, Georgia State Highway Planning Engineer

In this study, a comparison is made of the relative efficiencies of public transit vehicles and private automobiles in utilizing street space and transporting people. The operation of buses, streetcars, and automobiles was investigated in Washington, D. C., and trolley coaches and automobiles in Atlanta, Ga. An additional phase of the study included the comparison of bus and automobile operation on a section of the Atlanta Expressway.

Three factors were considered in evaluating the efficiencies of the various modes of travel: the speed of the vehicle, the space occupied by the vehicle in the traffic stream, and the number of persons traveling per vehicle.

The results of the study show that buses operating in Washington, D. C., were 3.7 times as efficient as automobiles on downtown streets, 4.6 times as efficient in intermediate areas, and 2.6 times as efficient in outlying areas. Streetcar versus automobile operation was observed in the downtown area of Washington only, and it was found that streetcars were 1.8 times as efficient as automobiles.

Trolley coaches in Atlanta, Ga., were found to be 6.3 times as efficient as automobiles on downtown streets, 8.7 times as efficient in intermediate areas, and 6.3 times as efficient in outlying areas. A comparison of bus and automobile operation on the Atlanta Expressway indicated that buses were 7.2 times as efficient as automobiles.

MASS TRANSPORTATION is recognized as a highly desirable if not essential component of the overall transportation scheme in urban centers. In recent years public transit systems have experienced a decline in patronage, while the number of persons using privately owned automobiles has represented an increasingly larger percentage of the people transported over city streets. The overcrowding of streets is often attributed to this decline in public transit patronage, and fears have been expressed that cities, and more particularly their transit companies, are facing economic ruin unless the trend is arrested. Subsidization has been proposed, and in several cases, it has been resorted to as a means of maintaining public transit service.

The purpose of the street system is to serve the public, and decisions as to how this service can be most satisfactorily and economically provided rest with the highway user, taken collectively. The increased use of private automobiles at the expense of transit patronage is merely a registration of highway-user attitudes. However, the expansion in automobile usage has thus far taken place on streets that have undergone only minor change or improvement through the years and the outlay for capital improvements of street systems has not approached the rate of growth of private automobile usage.

Large capital expenditures for street improvement are a prerequisite to the unabated growth of vehicular traffic. When the proportionate share of these costs is levied upon the private automobile user, the preference which he has thus far exhibited might be altered somewhat.

A thorough investigation of the economics of public transit versus private automobile usage of city streets would be very far-reaching and would cover all of the various aspects of user costs, convenience and comfort factors, time savings or losses, and other operational economies. As one phase of an economic investigation, the present study has for its purpose a comparison of the relative efficiencies of public transit vehicles and private automobiles in utilizing street space and transporting people.

A Standard for Measuring Efficiency

Any measure of efficiency must consider the space occupied per person in the moving traffic stream, and the length of time that the space is occupied in traveling a given distance. Space per person may be expressed as the space occupied in the traffic stream by a vehicle of a particular type, divided by the number of persons carried by the vehicle. The length of time that the space is occupied may be determined by dividing the distance traveled by the overall speed of the vehicle. Thus, for comparing efficiencies of different modes of transportation, there are three elements to be considered for each type of vehicle if conclusive results are to be obtained. These are the speed of the vehicle, the space occupied by the vehicle in the traffic stream, and the carried load. These three variables were investigated for automobiles, buses, and streetcars in the Washington, D. C., metropolitan area, and for automobiles and trolley coaches in Atlanta, Ga.

Conclusions

The investigations reported here are exploratory in nature, and the interpretations placed upon the results are based on operating conditions as they actually occurred during the periods studied. Recognition might well be taken of the fact that operating conditions are subject to change and that the rates at which people are transported in various types of vehicles do not in any sense represent the maximum number of people that could be transported under hypothetical operating conditions. For example, in no instance was the potential capacity of the streetcar-track lane fully utilized by public transit vehicles although the routes studied were the most heavily traveled in the city; and one site, with 113 streetcars in one hour, approached what is considered the limit in numbers that can use a street in one direction. More people could have been transported by streetcars without any further encroachment upon the street space, but this could happen only if more people wished to avail themselves of this mode of transportation, and if more streetcars were made available for their use.

In the case of automobiles, the potential capacity of the street space was more fully utilized by vehicles at most of the study sites than was the case for streetcars. Automobiles were operating at only one-third their passenger capacity, however. To hypothesize that more streetcars could operate over a single track (if there were people to ride on them) merely invites a parallel hypothesis that more people could band together as group riders in automobiles if they should so choose. For current operations, comparisons as made in this article would seem to be founded on as firm a basis as any that might be conceived. On this premise the following conclusions are drawn for peak-hour operations.

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[1] This article was presented at the 36th Annual Meeting of the Highway Research Board, Washington, D. C., January 1957.

1. In Washington, D. C., buses were 3.7 times as efficient as automobiles on downtown streets, 4.6 times as efficient in intermediate areas, and 2.6 times as efficient in outlying areas. On a freeway, buses in Atlanta, Ga., were 7.2 times as efficient as automobiles.

2. In Atlanta, trolley coaches were 6.3 times as efficient as automobiles on downtown streets, 8.7 times as efficient in intermediate areas of the city, and 6.3 times as efficient in outlying areas.

3. In Washington, streetcars were 1.8 times as efficient as automobiles on downtown streets.

4. The efficiencies of public transit vehicles in Washington, as derived in this investigation, cannot be compared directly with those in Atlanta because of differences in the operation of automobiles in the two cities. Automobiles in Washington traveled faster and carried more passengers than automobiles in Atlanta.

Bus Operation on City Streets

The investigation of bus operation included studies of travel speeds and of passengers carried by buses in normal operation during various periods of the day, and of the effect that buses have on street capacity. The speed and loading studies covered a number of bus lines in the Washington metropolitan area. Observers posted on buses compiled complete records showing the number of persons on board between stops, the number of persons boarding and alighting at each stop, the travel time between stops, the length of time spent in loading and unloading operations, and the cause and duration of delays.

The study extended over a period of 5 days, Monday through Friday, during the spring of 1950. Information was obtained on 283 bus trips (one way) divided about equally between the morning rush period, the afternoon rush period, and the off-peak or base period. Thirteen operating routes were studied.

Average carried load

The average number of passengers carried by buses during peak and off-peak periods are summarized in table 1. These buses were inbound during the morning peak period and outbound during the afternoon peak. The number of persons shown is the average per bus, weighted according to passenger-miles traveled.

Operating speed

The travel time for buses and automobiles during peak and off-peak periods is also shown in table 1. The data include minor traffic delays and delays resulting from traffic signals, and they represent the travel time for vehicles that were inbound during the morning peak period and outbound during the afternoon peak.

Travel time for automobiles was determined by driving a test car over the routes included in the study of buses. On each of these routes, nine trips were made with a test car driven at a speed which the driver thought was representative of the average speed of traffic. Three trips were made during the morning rush hour (not necessarily on the same day), three during the afternoon rush period, and three during the off-peak period.

Space occupied

In the study of the effect that buses have on street capacity, the maximum rates of traffic flow (possible capacities) for selected intersections in the Washington metropolitan area were determined by counting the number of vehicles entering each intersection from one of its approaches when the traffic demand was equal to or greater than the capacity of that approach. The traffic count for each signal cycle was classified as to the extent of interference by buses; that is, no interference, interference by one bus, by two buses, and so forth. The capacity of the intersection approach without bus interference was compared with its capacity with bus interference, and a determination was made of the number of automobiles displaced by one bus in the traffic stream. Satisfactory data were obtained for 16 intersection approaches—11 on major streets and 5 on minor streets. This study was conducted in June 1953.

The amount of space in a traffic stream that a bus occupies varies with the street width, the number of passengers loading and unloading, the gradient of the street, parking conditions, and a number of less important factors. The 11 major streets on which studies were conducted ranged in width from 40 to 60 feet. All of the intersection approaches except one had near-side bus stops and the one exception had a far-side stop. None of the study sites was in the central business district although some were on the fringe of the downtown area. Most of the locations were in intermediate-type areas with a few being in the outlying suburban districts. Parking was not permitted during rush hours on any of the major streets during the course of the studies and all bus traffic followed a straight course (no turns at intersections).

The interchange of passengers at the bus stops was only moderately heavy. Studies were conducted during morning and afternoon hours of peak traffic movement, and during the off-peak periods as well. With such a notable variation in the conditions at the various study sites, a corresponding variation in results was to be expected. On the average major street, one bus was found to have a displacement equivalent to 3.3 automobiles, as shown in table. 1. The range in the passenger-car equivalent of one bus was from 2.1 to 6.0. On the 5 minor streets, the average bus was found to be equivalent to 1.6 automobiles with a range from 1.0 to 1.9. An appreciable difference was observed in the bus equivalent between morning and afternoon study periods on the major streets, the former being about one and one-half times the latter on an average.

The disparity between the morning and afternoon values is presumed to be a result of differences in the conditions at the study sites and not a result of any difference in the characteristics of bus operations for the two time periods. The average of the values for morning and afternoon travel on major streets is used in subsequent comparisons of vehicle efficiencies.

For automobiles, the space per vehicle in the traffic stream has been taken as unity for purposes of this analysis. The other two variables—travel time and the average carried load—were determined by field investigation. The average number of persons per automobile was 1.97.

Relative efficiencies

Table 2 compares automobiles with buses in their travel time, carried load, space per vehicle in the traffic stream. and relative efficiencies. Travel time, carried load, and space per vehicle as shown in table 2 under the heading of "major streets" are average values for the morning peak hour in the inbound direction and the afternoon peak hour in the outbound direction. The relative efficiencies for the two types of transportation, automobiles and buses, are shown using the automobile as unity as a basis for comparison. In the down-

Table 1.—Average number of passengers carried by buses, average travel time for buses and automobiles, and average number of automobiles displaced by one bus

Variables	Time of day			
	Morning peak period (inbound)	Afternoon peak period (outbound)	Peak-hour average (inbound and outbound)	Off-peak period average (inbound and outbound)
Average number of persons per bus:				
Downtown areas	30.1	48.5	39.3	28.5
Intermediate areas	47.1	53.4	50.2	31.4
Outlying areas	25.6	36.0	30.8	16.8
Average travel time for buses:				
Downtown areas ... minutes/mile	10.1	9.2	9.6	9.6
Intermediate areas ... do	6.2	6.5	6.4	5.8
Outlying areas ... do	5.2	5.4	5.3	4.3
Average travel time for automobiles:				
Downtown areas ... do	5.8	6.2	6.0	6.2
Intermediate areas ... do	3.9	3.9	3.9	3.7
Outlying areas ... do	2.8	3.1	3.0	2.9
Number of automobiles displaced by one bus:				
Major streets	3.9	2.7	3.3	----------
Minor streets	1.6	1.7	1.6	----------

Table 2.—Relative efficiencies of automobiles and buses in the utilization of street space and movement of people during peak hours of traffic on major streets and on an urban freeway

Variables	Major streets			Urban freeway
	Downtown areas	Intermediate areas	Outlying areas	
Travel time (minutes per mile):				
Automobiles	6.0	3.9	3.0	1.8
Buses	9.6	6.4	5.3	2.4
Carried load (persons per vehicle):				
Automobiles	2.0	2.0	2.0	1.7
Buses	39.3	50.2	30.8	27.9
Space per vehicle (automobile units):				
Automobiles	1.0	1.0	1.0	1.0
Buses	3.3	3.3	3.3	1.7
Relative efficiency:				
Automobiles	1.0	1.0	1.0	1.0
Buses	3.7	4.6	2.6	7.2

town area, 39.3 passengers in one bus occupy as much space in the traffic stream as 6.6 passengers in 3.3 automobiles, a ratio of 5.95 to 1 in favor of the bus. These bus passengers occupy their space 1.62 times as long as the automobile occupants in traveling any given distance, so the efficiency of the bus is thereby reduced by the speed differential in the ratio of 1 to 1.62, or to 62 percent. The resultant efficiency of the bus in the downtown area during peak hours is 3.7 times that of the average automobile. In the intermediate area the bus is 4.6 times as efficient as the automobile, and in the outlying area it is 2.6 times as efficient as the automobile.

Bus Operation on Freeways

The average load and rate of travel for diesel-powered buses while operating in express service on a freeway were determined as one part of a study of public transit operation in Atlanta, Ga., in 1955. The bus route traverses the northeast leg of the Atlanta Expressway system, which is a freeway with full control of access. There are no intermediate bus stops between the point where the route enters the freeway in the residential district and the point of exit in the downtown area.

During the morning peak period, the average bus on the expressway carried 31.3 passengers and traveled at the rate of one mile in 2.5 minutes. During the afternoon peak period the average load was 24.5 passengers, and the rate of travel was one mile in 2.4 minutes. For the morning and afternoon periods combined, the average express bus carried 27.9 passengers and the average rate of travel was one mile in 2.4 minutes.

Automobiles using this portion of the Atlanta Expressway carried an average of 1.7 persons during the morning and afternoon peak periods and traveled at the rate of one mile in 1.8 minutes.

The number of buses in service on this route was too small to permit a reliable determination of the space occupied by a bus in a stream of expressway traffic. The Shirley Memorial Highway in Arlington, Va., a freeway in the Washington, D. C., metropolitan area, afforded a more satisfactory location for comparing the space occupied by an automobile with that occupied by a bus.

An extensive study by the Bureau of Public Roads revealed that for any given volume of traffic the average bus on the Shirley Memorial Highway occupies a time-gap in the traffic stream which is 1.7 times that for the average automobile. The study covered the operation of 658 buses and more than 75,000 automobiles.

Relative efficiencies

By using the travel time and carried load for automobiles and buses as found on the Atlanta Expressway, and the space occupied by a bus as found on the Shirley Memorial Highway, buses on freeways were found to be 7.2 times as efficient as automobiles in the utilization of freeway space and in the transport of people. The data are summarized in the last column of table 2.

Trolley-Coach Operation

The procedures used in making studies of bus operation on major streets in Washington, D. C., were employed in a study of trolley coaches in Atlanta. The study was made in February and March of 1955 and the results are summarized in table 3. The results of studies at 11 intersections are included. The similarity between the displacement of trolley coaches on major streets in Atlanta and of gasoline-powered buses on major streets in Washington, as shown in table 1, is very striking. Trolley coaches as well as buses seem to reduce the capacity of major streets to a considerably greater extent during the morning period of peak traffic movement than during the afternoon peak. It would seem to be more than happenstance that both buses and trolley coaches were found to contribute more to congestion in the morning than during the afternoon, but the reason for this is not clear. As in the case for buses, the average of the morning and afternoon values for the space occupied by trolley coaches on major streets is used in comparing vehicle efficiencies.

At the time that trolley coaches were carrying the number of passengers shown in table 3, automobiles using the same streets in Atlanta were carrying an average of 1.7 people. The average travel times for automobiles on these same streets are shown in table 3.

Relative efficiencies

The relative efficiencies of trolley coaches and automobiles, based on the three variables of travel time, persons per vehicle, and space occupied per vehicle, are shown in table 4. The figures given are average values for morning and afternoon peak periods. In the downtown area, for example, trolley coaches are shown as being 6.3 times as efficient as automobiles. Their speed is 0.7 that of automobiles; their load is 28.7 times that of automobiles; and their space in the traffic stream is 3.3 times that of an automobile. In the intermediate area the trolley coach is 8.7 times as efficient as the automobile, and in the outlying area the relative efficiency of the trolley coach is 6.3, the same as in the downtown area.

Streetcar Operation

The procedure for studying streetcar operation differed in several respects from that employed for buses and trolley coaches. The major difference, however, was that vehicles and passengers were counted as they passed fixed points along the routes rather than by having observers on the vehicles. The majority of the study sites chosen were in the downtown business district, with the remainder being in the area immediately adjacent thereto. Study sites were selected at loading platforms where automobiles and streetcars were each allotted a specific portion of the street separate and distinct from that available to the other. Attempt was made to choose locations where conditions were most highly favorable for public transit operations,

Table 3.—Average number of passengers carried by trolley coaches, average travel time for trolley coaches and automobiles, and average number of automobiles displaced by one trolley coach

Variables	Time of day			
	Morning peak period (inbound)	Afternoon peak period (outbound)	Peak-hour average (inbound and outbound)	Off-peak period average (inbound and outbound)
Average number of persons per trolley coach:				
Downtown areas	50.4	47.0	48.7	28.5
Intermediate areas	51.7	52.5	52.1	28.3
Outlying areas	38.9	40.1	39.5	19.4
Average travel time for trolley coaches:				
Downtown areas.....minutes/mile	10.7	12.2	11.4	9.8
Intermediate areas.....do	5.6	6.4	6.0	5.2
Outlying areas.....do	4.2	4.3	4.3	3.7
Average travel time for automobiles:				
Downtown areas.....minutes/mile	8.2	8.6	8.4	6.7
Intermediate areas.....do	5.3	5.9	5.6	4.2
Outlying areas.....do	3.7	4.2	3.9	3.2
Number of automobiles displaced by one trolley coach:				
Major streets	4.0	2.6	3.3	--------
Minor streets	3.8	3.8	3.8	--------

Table 4.—Relative efficiencies of automobiles and trolley coaches and automobiles and streetcars (downtown areas only) in the utilization of street space and movement of people during peak hours of traffic on major streets

Variables	Automobiles and trolley coaches operating in—			Automobiles and streetcars operating in downtown areas
	Downtown areas	Intermediate areas	Outlying areas	
Travel time (minutes per mile):				
Automobiles	8.4	5.6	3.9	6.0
Trolley coaches	11.4	6.0	4.3	----------
Streetcars	----------	----------	----------	11.0
Carried load (persons per vehicle):				
Automobiles	1.7	1.7	1.7	2.0
Trolley coaches	48.7	52.1	39.5	----------
Streetcars	----------	----------	----------	50.8
Space per vehicle (automobile units):				
Automobiles	1.0	1.0	1.0	1.0
Trolley coaches	3.3	3.3	3.3	----------
Streetcars	----------	----------	----------	7.6
Relative efficiency:				
Automobiles	1.0	1.0	1.0	1.0
Trolley coaches	6.3	8.7	6.3	----------
Streetcars	----------	----------	----------	1.8

that is, along the most heavily traveled and highly patronized transit routes. A total of 10 sites were studied and, with one exception, these sites happened to be on important automobile routes. However, transit traffic was the primary consideration governing their selection.

Studies were conducted during morning and afternoon hours of peak traffic on four weekdays in April 1952. Information collected consisted of a simple count of vehicles in each category and the number of persons in each vehicle. Traffic in the direction of the heavier movement only was counted and the field data were summarized for each signal cycle (80 seconds). From this study a comparison was made of the space occupied by streetcars and automobiles and the load carried by vehicles of each type.

Travel time

For the purpose of measuring streetcar travel time, three streets were selected and these were either within or adjacent to the central business district of Washington.

Travel time for streetcars was determined by observers working in pairs on the three streetcar lines. The observers were stationed several blocks apart and each recorded the exact time of day that every streetcar passed his station, together with the identification number painted on the side of the car. Later, the records for the two observers were compared, and the elapsed time for each streetcar to traverse the known distance between the observation points was computed. Data were collected for 750 streetcars covering approximately 450 vehicle-miles of travel. The average travel time during periods of peak traffic was found to be 11.0 minutes per mile.

The rate of travel for automobiles, as measured in the study of bus operation and as shown in table 1, is used in comparing automobile and streetcar operation.

Average load carried

A summation of data for the heaviest hour of traffic at each of the 10 study sites shows that 690 streetcars and 9,126 automobiles were observed. The 690 streetcars transported 35,070 passengers, whereas automobiles carried a total of 17,932 passengers including drivers. Average carried loads were 50.83 persons for streetcars as against 1.97 for automobiles. The average streetcar carried 25.8 times as many people as the average automobile.

Space per vehicle

The space occupied by the average vehicle of each type may be measured by dividing the number of vehicles per hour by the width of street available to that type of vehicle. At the average site, 4.7 streetcars per hour per foot of width availed themselves of the space allotted to them, that is, the width of the car-track lane to the center of the street and including the loading platform. The average number of automobiles utilizing the space between the curb and platform was 35.9 per hour per foot of street width. For operating conditions as they occurred during the week of the study, the average streetcar occupied 7.6 times as much space in the traffic stream as the average automobile.

Relative efficiencies

The procedure for calculating the relative efficiencies of automobiles and streetcars is the same as that followed in comparing automobiles and buses. Relative efficiencies of streetcars, as compared with automobiles, together with values of the variables used in developing these efficiencies, are shown in the last column of table 4. The streetcar is shown as being 1.8 times as efficient as the automobile in downtown Washington. Data were not obtained for other areas of the city.

Efficiencies of Public Transit Vehicles and Automobiles Summarized

The operation of each of three types of public transit vehicles has been compared with automobiles that were using the same or similar streets during approximately the same period of time. The three variables used in the comparison were travel time, number of passengers, and space occupied in the traffic stream. The resulting efficiencies, based on the three variables, are summarized in the upper portion of table 5.

Figures in table 5 cannot be used to compare transit vehicles in one city with transit vehicles in another city because the automobiles, which were used as a standard for comparison, varied in their operation on different types of streets, and more particularly between cities. Automobiles in Atlanta, for example, traveled slower and carried fewer passengers than automobiles in Washington. It is primarily for this reason that trolley coaches in Atlanta show a higher efficiency than buses in Washington. The inference should not be drawn that trolley coaches would be almost twice as efficient in Washington as buses.

As another precaution, table 5 should not be interpreted as meaning that automobiles could be substituted for transit vehicles in the numbers shown in the table with the result that the same total number of persons could be transported by automobile alone as are presently being moved by transit vehicles and automobiles combined.

Fewer people could be transported in automobiles alone than can be moved on a street by automobiles in combination with transit vehicles. but this smaller number of people would reach their destinations in a shorter length of time. If the concern is with

Table 5.—Relative efficiencies of various types of vehicles in the utilization of street space and movement of people

Mode of travel	Downtown areas	Intermediate areas	Outlying areas
RELATIVE EFFICIENCIES CONSIDERING TRAVEL TIME, PERSONS PER VEHICLE, AND SPACE PER VEHICLE			
Automobiles	1.0	1.0	1.0
Buses (major street operation)	3.7	4.6	2.6
Buses (freeway operation)	----------	7.2	----------
Trolley coaches	6.3	8.7	6.3
Streetcars	1.8	----------	----------
RELATIVE AMOUNT OF SPACE OCCUPIED BY ONE PERSON			
Automobiles	1.00	1.00	1.00
Buses (major street operation)	.17	.13	.21
Buses (freeway operation)	----------	.10	----------
Streetcars	.30	----------	----------
RELATIVE EFFICIENCIES CONSIDERING PERSONS PER VEHICLE AND SPACE OCCUPIED PER VEHICLE			
Automobiles	1.00	1.00	1.00
Buses (major street operation)	5.95	7.60	4.67
Buses (freeway operation)	----------	10.00	----------
Streetcars	3.38	----------	----------

absolute numbers of persons that can be moved on a street without regard to rate of travel, then the transit vehicle enjoys a much greater advantage than is reflected in table 5. The space in the traffic stream which a person occupies while traveling in various types of vehicles is a more reliable measure of the number of people that can be moved past a point by vehicles of each type. The middle portion of table 5 shows the relative amount of space in the traffic stream a person occupies while traveling in automobiles, buses, and streetcars.

Translated into relative efficiencies of street-space utilization, the reciprocals of the figures for space occupied by one person are shown in the lower portion of table 5. As previously mentioned, a direct comparison cannot be made between trolley coaches and buses or streetcars because trolley coach operation is related to automobile operation in Atlanta, whereas bus and streetcar operation is related to automobile operation in Washington.

Acknowledgments

This investigation is a product of the joint efforts of several agencies. The studies of travel time and passengers carried by trolley coaches and automobiles in Atlanta were performed by personnel of the Georgia State Highway Department, with the cooperation of the Atlanta Transit Company in the selection of routes and the furnishing of riders' passes to those engaged in the study. Field work for a determination of the space occupied by trolley coaches in the traffic stream was performed by the Traffic Engineering Department of the City of Atlanta.

Field work for the bus study in Washington, D. C., was performed by Bureau of Public Roads personnel with the assistance of personnel furnished by the Capital Transit Company. Participation by that Company included the services of five observers under the supervision of the Company's Research and Planning Department. The Company also participated in establishing procedures for the study and provided riders' passes to all engaged in the study. The Washington-Virginia-Maryland Coach Company of Arlington, Va., supported the study by furnishing free fares to observers operating on their lines.

Highway Statistics, Summary to 1955

Highway Statistics, Summary to 1955, a general historical summary of factual information dealing with highways, their use, and financing, is now available. The new publication may be purchased from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at $1 a copy.

This 150-page bulletin brings together under one cover a comprehensive statistical review of highway development in the United States through 1955, and includes all of the data presented in a previous compendium, "Highway Statistics. Summary to 1945," and most of the material published since then in the "Highway Statistics" annual bulletins.

The material is presented in four major groupings. The motor-fuel section includes analysis of motor-fuel consumption, tax rates, and tax receipts. The section on motor vehicles includes tables on motor-vehicle registrations and operators' licenses, their fee schedules, and the revenues received therefrom and from motor-carrier taxes; also included in this section are travel, loading, and speed data. The highway finance section covers the disposition of highway-user imposts, receipts and expenditures for highways, and highway debt; because of the interest in the subject, data for toll facilities are segregated. Although the local finance historical series are not presented separately in this bulletin, combined State and local government finance data are reported in several series of tables. The mileage section reports road and street mileage existing, and the mileage built each year, classified by system and by type. The section on Federal aid includes tables on Federal excise taxes and on Federal-aid funds, construction, and system mileage.

U. S. GOVERNMENT PRINTING OFFICE: 1957

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PUBLICATIONS

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Parking Guide for Cities (1956). 55 cents.
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Tire Wear and Tire Failures on Various Road Surfaces (1943). 10 cents.
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MAPS

State Transportation Map series (available for 39 States). Uniform sheets 26 by 36 inches, scale 1 inch equals 4 miles. Shows in colors Federal-aid and State highways with surface types, principal connecting roads, railroads, airports, waterways, National and State forests, parks, and other reservations. Prices and number of sheets for each State vary—see Superintendent of Documents price list 53.
United States System of Numbered Highways. 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

Bibliography on Automobile Parking in the United States (1946).
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Bibliography on Roadside Control (1949).
Express Highways in the United States: a Bibliography (1945).
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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF AUGUST 31, 1957

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES 1/	ACTIVE PROGRAM											
		PROGRAMMED ONLY			CONTRACTS ADVERTISED, CONSTRUCTION NOT STARTED			PROJECTS UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$73,689	$35,109	$26,938	331.5	$12,740	$8,380	126.9	$74,331	$46,178	836.1	$122,180	$81,496	1,294.5
Arizona	34,659	19,060	17,247	117.4	5,143	3,912	64.9	26,941	22,694	159.6	51,144	43,853	341.9
Arkansas	51,028	32,461	23,549	472.7	15,489	9,224	140.1	46,360	30,677	511.5	94,310	63,450	1,124.3
California	156,907	30,800	19,039	218.6	17,411	13,437	17.3	411,602	215,384	336.7	459,813	247,860	572.6
Colorado	51,429	20,516	14,781	166.3	9,498	7,311	96.9	47,333	33,058	277.0	77,347	55,150	540.2
Connecticut	55,929	2,480	1,260	3.4	11,625	8,623	5.7	32,876	20,423	39.4	46,981	30,306	48.5
Delaware	31,956	2,530	1,293	14.7	2,538	1,481	10.0	15,052	8,306	62.1	20,120	11,080	86.8
Florida	60,033	37,651	25,927	427.7	16,830	13,029	35.3	43,459	24,973	226.8	97,940	63,929	689.8
Georgia	105,462	49,958	31,285	568.9	11,286	5,975	70.8	90,623	51,728	864.0	151,867	88,988	1,503.7
Idaho	48,377	18,025	13,089	144.8	1,952	1,426	18.7	18,573	12,879	202.1	38,550	27,394	365.6
Illinois	128,474	90,221	62,619	549.2	66,545	52,784	90.6	153,528	107,999	780.1	310,294	223,402	1,419.9
Indiana	146,797	24,193	13,187	147.0	15,966	8,270	184.3	50,802	32,024	234.5	90,961	53,481	565.8
Iowa	62,902	41,751	35,692	275.5	10,628	6,809	124.1	69,577	44,444	1,655.3	121,956	86,945	2,054.9
Kansas	56,024	42,289	34,856	746.9	7,327	4,721	119.6	51,346	31,618	1,374.3	100,962	71,195	2,240.8
Kentucky	93,196	8,112	4,140	81.6	10,490	6,057	49.2	51,232	34,924	260.7	69,834	45,121	391.5
Louisiana	69,437	43,589	27,192	212.0	14,072	8,873	75.8	53,166	29,336	317.8	110,827	65,401	605.6
Maine	44,957	9,442	5,031	80.3	1,322	665	19.5	18,796	9,996	106.7	29,560	15,692	206.5
Maryland	29,209	29,950	17,431	87.0	15,252	10,188	18.1	63,270	43,312	215.5	108,472	70,931	320.6
Massachusetts	79,603	36,667	24,002	31.8	49,499	33,050	32.8	63,096	34,590	49.1	149,262	91,642	113.7
Michigan	115,868	66,917	51,161	446.6	32,602	18,239	75.1	111,198	76,560	601.0	210,717	145,960	1,122.7
Minnesota	74,620	15,933	13,694	176.4	4,830	3,288	38.1	102,986	72,005	1,651.9	123,749	88,987	1,866.4
Mississippi	48,144	30,118	21,238	566.6	22,090	18,047	129.5	51,608	32,736	821.5	103,816	72,021	1,517.6
Missouri	97,607	27,342	16,651	1,024.8	7,974	5,725	34.6	127,265	81,489	1,392.7	162,581	103,865	2,452.1
Montana	72,776	11,168	7,861	238.8	6,489	4,394	75.9	42,637	30,971	314.3	60,294	43,226	629.0
Nebraska	79,498	15,898	8,861	246.5	7,384	4,195	97.6	43,407	25,289	1,224.1	66,689	38,345	1,568.2
Nevada	43,146	13,483	12,601	46.0	984	823	14.0	21,547	18,670	190.1	36,014	32,094	250.1
New Hampshire	21,341	10,542	7,973	28.4	1,717	1,500	.6	21,083	13,705	68.6	33,342	23,178	97.6
New Jersey	103,923	9,498	5,484	65.6	23,859	19,227	11.8	58,322	37,534	49.8	91,679	62,245	127.2
New Mexico	39,348	3,733	2,527	39.2	3,704	2,790	51.2	37,873	31,716	225.5	45,310	37,033	315.9
New York	215,349	43,127	29,022	67.4	60,917	34,718	80.8	479,209	288,615	551.7	583,253	352,355	699.9
North Carolina	105,021	35,803	24,657	326.9	6,924	4,783	50.8	84,735	50,549	949.9	127,462	79,989	1,327.6
North Dakota	42,960	20,943	16,013	963.0	2,681	1,929	49.5	28,509	18,016	1,170.7	52,133	35,958	2,183.2
Ohio	107,765	100,680	68,910	176.1	31,476	21,768	45.4	198,055	131,594	285.1	330,211	222,272	506.6
Oklahoma	60,091	45,301	34,480	443.6	15,255	11,057	65.5	56,432	33,543	562.8	116,988	79,080	1,071.9
Oregon	48,948	10,564	7,685	39.2	9,540	7,350	42.0	44,537	33,073	262.1	64,641	48,108	343.3
Pennsylvania	155,455	90,675	58,397	194.0	75,637	59,630	93.1	213,344	131,256	363.5	379,656	249,283	650.6
Rhode Island	19,365	5,734	3,140	11.0	4,764	3,570	4.9	31,571	22,215	16.6	42,069	28,925	32.5
South Carolina	52,616	40,065	28,897	369.1	3,713	3,040	22.8	38,337	22,394	738.2	82,115	54,331	1,130.1
South Dakota	42,081	27,428	21,512	345.4	1,512	845	62.5	28,567	19,451	736.9	57,507	41,808	1,144.8
Tennessee	65,510	61,856	47,721	343.4	14,864	8,695	48.0	81,350	48,423	652.5	158,070	104,839	1,043.9
Texas	226,761	22,930	13,876	350.6	59,746	45,083	283.5	179,752	118,317	1,649.0	262,428	177,276	2,283.1
Utah	42,442	15,218	12,496	114.7	1,523	1,135	2.0	18,584	14,632	211.5	35,325	28,263	328.2
Vermont	25,433	2,113	1,161	23.2	2,981	2,683	1.9	18,802	12,889	57.5	23,896	16,733	82.6
Virginia	58,400	48,283	37,002	170.1	24,164	16,076	64.6	54,964	36,125	277.9	127,411	89,203	512.6
Washington	68,897	15,821	10,978	123.1	3,453	2,610	31.3	53,853	37,798	294.2	73,127	51,386	448.6
West Virginia	48,819	47,057	33,505	86.0	3,507	1,784	13.6	32,369	16,614	78.6	82,933	51,903	178.2
Wisconsin	101,790	27,015	17,965	308.2	10,160	6,252	84.0	65,772	38,781	512.6	102,947	62,998	904.8
Wyoming	23,082	13,116	10,343	48.9	8,674	7,564	48.2	42,380	33,213	367.3	64,170	51,120	464.4
Hawaii	7,539	7,260	3,630	15.6	71	36		4,138	1,985	7.9	11,469	5,651	23.5
District of Columbia	30,552	7,990	6,552	2.9	2,913	1,801	1.9	17,603	13,856	2.0	28,506	22,209	6.8
Puerto Rico	16,690	3,835	1,390	9.0	1,146	557	1.8	18,899	8,967	59.6	23,880	10,914	70.4
Alaska	13,645	4,238	4,234	58.8	3,042	3,042	113.7	8,668	7,160	275.4	15,948	14,436	447.9
TOTAL	3,655,550	1,476,488	1,040,175	12,146.4	755,909	528,451	3,040.8	3,800,319	2,394,684	25,132.3	6,032,716	3,963,310	40,319.5

1/ Includes funds authorized for the fiscal year 1959 apportioned August 1, 1957.

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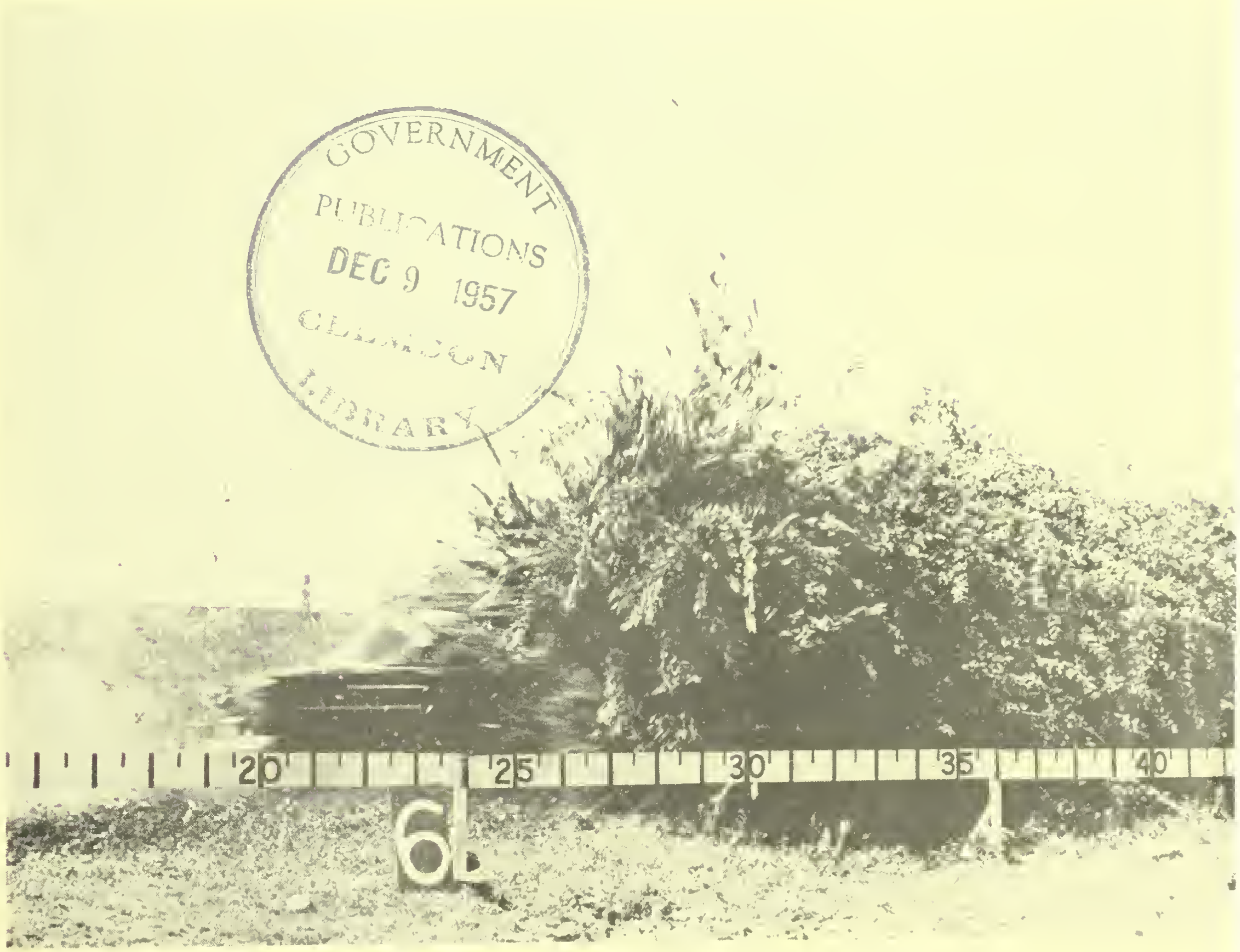

Testing the effectiveness of multiflora rose hedges as a crash barrier.

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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C. M. Billingsley, ***Editor***

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The printing of this publication has been approved by the Director of the Bureau of the Budget, Mar. 17, 1955.

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Crash-Barrier Tests on Multiflora Rose Hedges

By **RUSSELL R. SKELTON,**[1]
Professor of Civil Engineering,
University of New Hampshire

A number of colorful stories have appeared in print during the past few years about the possible use of multiflora rose hedge as a safe crash barrier in the median of a divided highway. They have sprung to some extent from the limited tests described in this article. The facts show that a multiflora rose hedge can safely stop an automobile, moving at 50 miles an hour, in a distance of about 75 feet on the path of travel. At an approach angle of 20 degrees, under such circumstances, a hedge 40 feet wide would be needed to stop the vehicle. However, even on the Interstate System, medians will usually provide less than this required distance—and vehicles quite commonly will be traveling more than 50 miles an hour.

The author presents in this article only the conclusions that may properly be drawn from this test of a multiflora rose hedge as a crash barrier. Other considerations indicate that plantings in medians as crash barriers are likely to be fully and economically useful only under special conditions. Economic factors involved in the use of crash-barrier median hedges were not studied in this test, but they are important. The initial planting cost and the cost of pruning and replacement may not be insignificant. Hedges, particularly of the thorny multiflora rose, accumulate a great deal of trash, paper, and other litter, which may be more expensive to remove than on a grassy median. Medians are often used, in the Northern States, for storage of snow plowed off the pavement, and the hedges may necessitate a different snow-removal practice.

One of the cited merits of median plantings is the elimination of headlight glare from traffic on opposing roadways. There seems little doubt as to the validity of this point. However, other highway-user reactions need to be considered. The hedges will cut off headlight glare, but they will also cut off the view of the opposing roadway and the scenery beyond it. Plantings will be attractive in summer, but less so in winter. The monotonous "tunnel" effect of continuous plantings may affect driver behavior.

All of these possible advantages and disadvantages should be weighed. Several States now have experimental plantings, and their performance, costs, and effects should be carefully studied.

PLANNED LANDSCAPING, using shrubs, not only could make a highway more attractive and less monotonous but these same shrubs could save lives. Judicious planting of several varieties of shrubs in median strips would not only reduce headlight glare but the same shrubs could act as a barrier or buffer between opposing streams of traffic. Also, such plantings could prevent disastrous impacts into bridge abutments or center piers.

Considering the recent trend in highway fatalities and injuries, every idea, every device, and every plan that offers any possible chance of saving lives should be critically studied. For that reason the Bureau of Public Roads and the University of New Hampshire became interested in the idea that a barrier of shrubs could serve as a living guard rail. The university, in 1954, proposed a program for testing the effectiveness of multiflora rose hedges as crash barriers, and the Bureau approved a sponsored research grant for the study.

The firm of Motor Vehicle Research, Inc., of South Lee, N. H., was retained as a consultant in the operation of the project. The firm's director, Andrew J. White, had previously run several crash tests on multiflora rose hedges. His earlier experiences were largely responsible for the Bureau's and university's interest in conducting a larger and more comprehensive testing program to determine the effectiveness of multiflora rose hedges as highway crash barriers under the impact of an automobile.

Of immediate and specific interest in the test program were: (1) the distance required to stop the vehicle after striking the hedge at various speeds and angles of approach, (2) the magnitude of deceleration, (3) the performance characteristics of the hedge and vehicle, (4) the extent and character of damage to the vehicle and hedge, and (5) a comparison of the stopping distance and hedge performance for the summer and winter tests.

A continuous single row of multiflora rose hedge, 14 years old and averaging 9 feet high and 10 feet wide, was used for the crash tests. A schedule of 11 field tests per season was planned, to include 5 angles of approach ranging from 5° to 90° (head-on) and from 1 to 3 speeds between 30 and 50 miles per hour for each angle of approach. The proposed schedule of tests had to be drastically reduced because the length of hedge destroyed by each run was considerably greater than was initially expected. Nine tests were run in July and three were run in December of 1955

Consideration of Findings

The following findings are based upon the results of tests obtained under the conditions described later in the report:

1. The effective travel length within the multiflora rose hedge required to stop a passenger car for a given speed, without the use of power or brakes, was dependent upon the age of the hedge bushes, their density, and their spacing.

2. The multiflora rose hedge proved to be an effective barrier for stopping a passenger car, provided the width was sufficient to prevent the vehicle from passing through the hedge. For the vehicle to be stopped within the hedge at a speed of 50 miles per hour, without the use of brakes or power, the minimum required effective length of hedge on the path of travel was 75 feet.

3. Angular approach and contact with the hedge did not deflect the car away from the barrier. Once the car was turned into the hedge, the angle-approach crash appeared to require about the same effective length of hedge to stop the car as was required by the head-on crashes.

4. The hedge provided a tough, resilient yielding barrier and permitted the forces of impact to be absorbed so gradually that the maximum deceleration was well within human tolerance.

5. The performance and effectiveness of the hedge in stopping the car were about the same for the winter tests as for the summer tests.

6. It was estimated that 25 percent of the bushes were pulled out in a crash. The remaining 75 percent were not critically damaged and probably will grow again to almost full effectiveness within a few years.

7. The test vehicle was not damaged except for very minor scratches.

8. During the destructive testing, a recurring phenomenon indicated that a sizable portion of the crash energy was absorbed after a mass of loose sheared bushes had accumulated ahead of the car. The shearing of the bushes ceased

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[1] This article was presented at the 36th Annual Meeting of the Highway Research Board, Washington, D. C., January 1957.

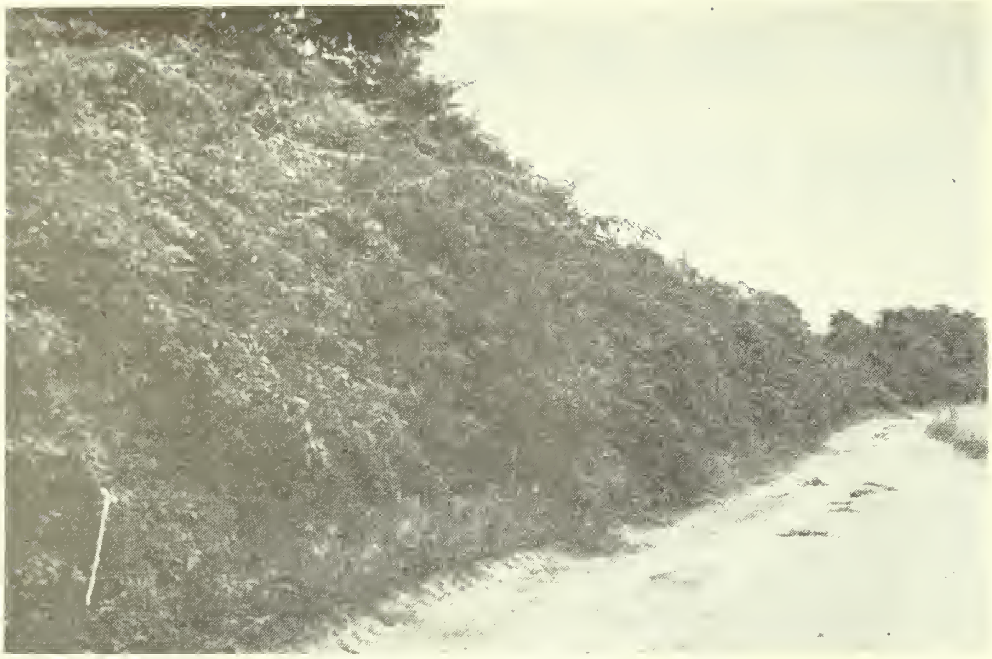

Figure 1.—Typical section of multiflora rose hedge and parallel road.

when the moving mass bent over the forward bushes.

9. Conclusions drawn from the test driver's observations and reactions are summarized as follows: (a) After the car crashed the barrier and was enmeshed in the hedge, the driver had no steering control of the vehicle; (b) the forces experienced during the stopping period seemed no more severe than an extreme emergency stop.

On the basis of the test data, it is estimated that the total width of hedge required for stopping passenger cars within the hedge, when traveling at speeds not exceeding 50 miles per hour and without brakes or applied power, would be about as follows:

Angle of approach, degrees	*Width of hedge, feet*
5	25
10	30
20	40
30	50
90 (head-on)	80

Divided highway medians should have about 10 feet clear space adjacent to each roadway pavement as emergency shoulders. Total median width, if plantings are used as crash barriers, would thus have to be about 20 feet wider than the hedge widths recommended. It might be pointed out that the optimum minimum recommended median width for the Interstate System, in flat and rolling topography in rural areas, is 36 feet.

A hedge insufficiently wide to prevent a complete breakthrough would nevertheless slow the crashing vehicle considerably. Such an advantage, however, might largely be nullified by the "surprise" effect on motorists in the opposing roadway, who would have practically no advance warning that a vehicle was crossing the median.

Characteristics of Rosa Multiflora

This plant was first described by Thunberg, a Swedish botanist, in 1784. The plant is of Asiatic origin and many varieties were found in Japan, Korea, and along the China coast. The plant was first introduced to Europe and America about 1875.

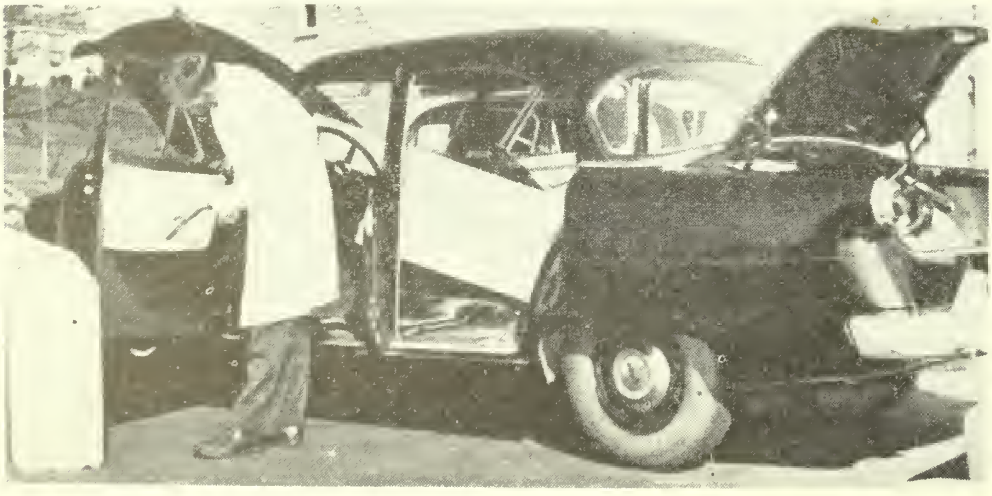

Figure 2.—Test car being equipped with plastic foam for driver protection.

Rosa Multiflora (japonica) is described as "a vigorous, dense shrub with long, arching, moderately thorny canes often exceeding 12 feet in height and width, and producing enormous quantities of single, white, blossoms in many-flowered pyramidal corymbs, in late May or early June . . . . Test plantings have proved that *R. multiflora* is adapted to all but extreme northern and southern states, and that when planted at one-foot intervals in a soil of average fertility, either alkaline or acid, will grow into an effective barrier against all livestock, except poultry, within three to six years." [2]

The multiflora plant, as tested, appeared to have a rather shallow but somewhat massive root system. The root system was confined to an area smaller than the total spread of the canes from one plant. Several of the roots encountered were as large as the largest cane and measured 6 feet in length. The root system seemed to provide very firm anchorage.

The plant does not produce suckers, hence a hedge may be controlled by pruning; however, birds carry the seeds, and seedling plants frequently spread rapidly on adjacent land. A hedge of the plants, when planted on 3-foot centers, will form an impenetrable barrier within 8 years. The canes branch and spread laterally, intertwining to form a dense barrier which is shock resistant.[3] It is this springlike characteristic that makes the Rosa Multiflora a potentially ideal plant for use along our highways as a living guard fence.

The Test Site

The continuous single row of mature multiflora rose hedge used for the tests was located in the vicinity of Manchester, Conn. The hedge was 14 years old, and was donated to the university by the owner for test purposes. The average height was 9 feet and the average width was 10 feet. The density of growth varied considerably in different sections, as did the individual cane diameters. The plants were spaced about 3 feet apart and there were few instances of seedling plants developing within the existing hedge.

A narrow earth road ran parallel with the hedge for its entire length. This road was used for the approach run for all tests. Figure 1 shows one section of the multiflora rose hedge and the adjacent road. The section shown contained 310 feet of hedge which was used for crash testing at various angles of approach. The extreme end was used for one high-speed winter test.

Test Car and Instrumentation

The test car, a 1952 Ford, 6-cylinder, 4-door sedan, was loaned to the university by the Bureau for the duration of the two-season field test period. It was stripped of all nonessential interior equipment that could possibly contribute to the injury of an occupant. The seats were removed and two front

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[2] *History of the Rose,* by Roy E. Shepherd. The Macmillan Co., New York, 1954, pp. 32-39.

[3] See footnote 2; also *The Multiflora Rose for Fences and Wildlife,* by Wallace L. Anderson and Frank C. Edminister. Leaflet No. 374, U. S. Department of Agriculture, Washington, D. C., Dec. 1954.

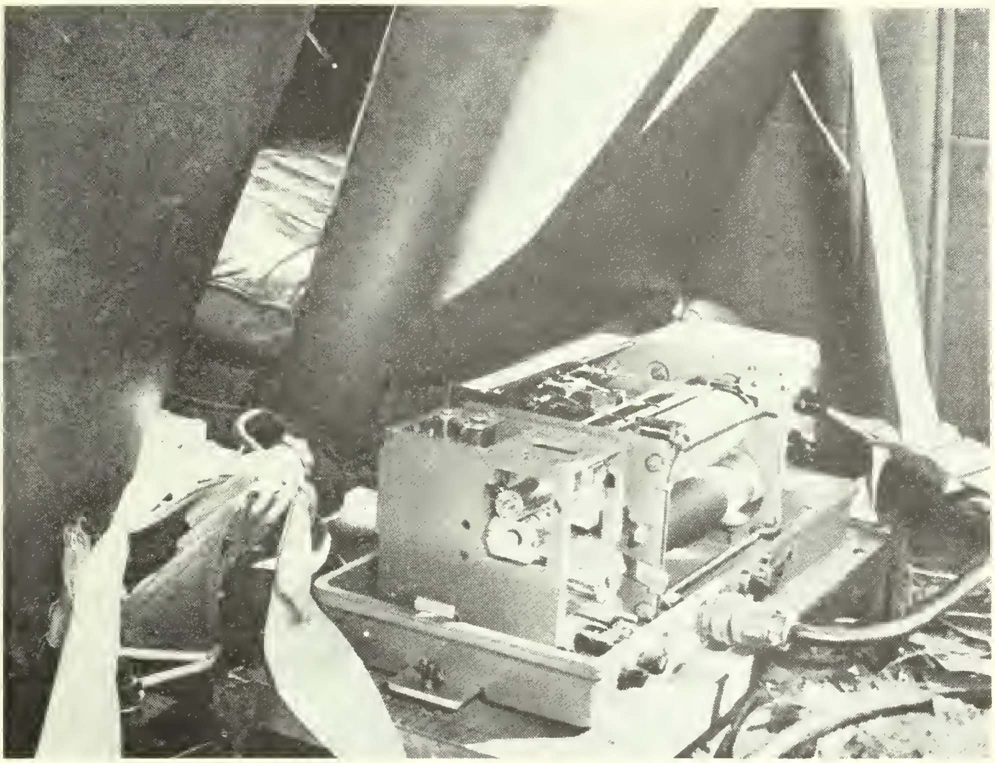

Figure 3.—Mounting of accelerometer in test car.

bucket-type seats were bolted to the floor. Two sets of 4-inch-wide seat belts and two sets of shoulder harness were fastened securely to the floor of the car. The dash assembly was fitted with a 6-inch covering of polystyrene plastic foam. Figure 2 shows the protective plastic foam being fitted to the interior.

A tachograph mounted on the firewall under the hood was coupled to the speedometer adapter gear. This instrument was used to obtain the top speed of the car during the approach run. The test car speedometer was calibrated by attaching a previously calibrated test wheel to the rear bumper and recording simultaneous readings at 10-miles-per-hour intervals up to 60 m.p.h. The test car speedometer readings were generally 6 to 10 percent higher than the true speed.

Since one of the principal objectives of the study was to determine the magnitude of deceleration, a motor-driven, constant-speed, three-component accelerometer was obtained for these tests. This instrument is shown in place with the cover removed in figure 3. The accelerometer weighed 18 pounds and measured 8 inches wide, 8 inches high, and 18 inches long.

The accelerometer was positioned on the centerline of the car and as near the actual center of gravity of the car as the front seats would permit. Actually the center of the instrument was 13⅞ inches to the rear of the center of gravity of the loaded car. In the vertical direction the center of the instrument was very close to the center of gravity of the car.

The accelerometer was equipped with a 115-volt, a. c., 60-cycle, electric motor and inverter. The 6-volt car battery was used as a source of power and heat for the styli and the case. Remote control switches were furnished for the operation and control of the motor and heaters. Wax-coated charts, 7 inches wide, having a 2-inch space for each component were used for recording the acceleration and deceleration over the full range of each stylus. The chart speed was 60 inches per minute.

The maximum deceleration in terms of gravity (g) for each direction was estimated and the manufacturer set each stylus to the specified range as follows: longitudinal direction, plus and minus 10 g; vertical direction, plus and minus 5 g; and lateral direction, plus and minus 5 g. The natural frequency was certified to be 15 cycles per second. Orifices were provided for air damping in all three planes. The degree of damping recommended and used on these tests was 80 percent, and the manufacturer calibrated and certified the instrument.

A single component recording, spring-driven accelerometer was placed in the spare tire well in the rear of the car. This instrument was positioned to record the acceleration in the vertical direction. The range of this instrument was plus or minus 1 g.

The test car hood and front fenders were given a heavy coat of white water-soluble paint prior to field testing so that the car could be more easily followed in the photographs. This covering also served to mark the points of severe contact with the bushes. Distinctive reference marks were placed on the vehicle to aid in the analysis of photographic film.

Photographic instrumentation included one spring-driven, 35-mm., wide-angle-lens camera set to run at a speed of 48 frames per second, and one power-operated 16-mm. camera set to run at 128 frames per second. The cameras were positioned perpendicular to the axis of the hedge on an elevated platform at a measured distance from a reference fence. They were adjusted to include the entire crash performance and a portion of the approach run. One additional spring-driven 16-mm. camera, operating at 64 frames per second, was used to obtain the general performance characteristics of the hedge and car.

A reference fence was erected parallel to the axis of the bushes for each test. This fence served as a base line for measurement and a photographic reference for film analysis. This reference fence is shown in the cover picture.

Test Procedure

Since the hedge consisted of a continuous single row of intertwined bushes, the tests were limited to head-on and small-angle crash tests. Because of physical limitations of the access road and the small width of the hedge, angles of approaches greater than 20 degrees were not made. In head-on tests, the car approached and crashed along the axis of the hedge at an established point. In angle tests, the car approached on a line marking an established angle with the hedge centerline and crashed the fringe of the hedge at a fixed point.

Before running a test, the point of contact, the centerline of the hedge, and dimensions of the hedge were measured in relation to the reference fence. When feasible, a bush count and the number and diameter of individual canes for several typical bushes were made. Representative cuttings were taken for water content determination. All tests were made on sections of the hedge unaffected by previous tests.

In general, the driver declutched the car approximately 20 feet ahead of the marked contact point. This standard procedure eliminated applied power, leaving only the momentum of the car to be considered in the analysis. The loss of speed in this short distance was usually of the order of 8 m.p.h. The roughness and lack of compaction of the approach road not only reduced the speed but introduced considerable pitching of the car in a vertical plane. The latter condition was more pronounced on low-speed tests.

The accelerometers were turned on when the test car started the approach run. All cameras were started when the car was about midway of the approach. Immediately after the car had stopped, the distance from the front bumper to the contact point was measured and recorded as the stopping distance. Offset distances from the reference fence to the tire tracks were made for record. A sketch was prepared showing the area and number of bushes damaged and the final position of the car in relation to the hedge and reference fence. Figure 4 is a sketch of a typical head-on crash and figure 5 is one of a 5° angle test. Also, following each test, the driver's observations concerning the test were noted. The test car was then removed from the hedge and examined.

Film Evaluation

All films were developed and reduced to 16-mm. positive prints for evaluation by a frame-by-frame method of analysis. A microfilm viewer having a large screen was used for the 48- and the 64-frames-per-second (fps) film analysis. The 128-fps film was analyzed

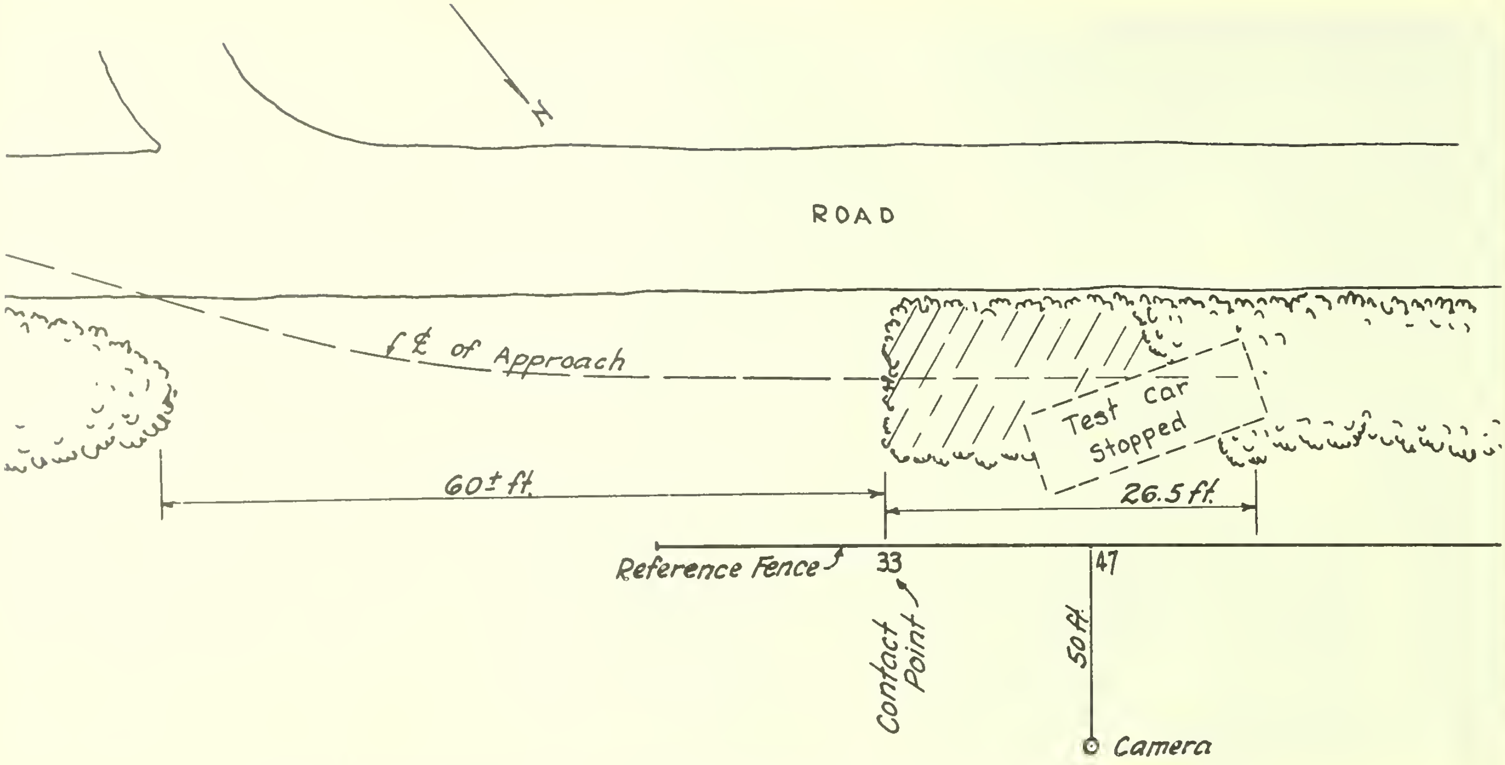

Figure 4.—Sketch of test No. 1 (head-on approach at 30.3 miles per hour contact speed).

by using a time-study projector. A sufficient number of check tests were made on several films using both pieces of equipment to determine whether the results obtained were in satisfactory agreement.

In the frame-by-frame analysis, the film was run until the test car came into view at a point in line with the reference fence. One of the several reference points on the car was selected and noted. The initial frame was recorded as zero and the reading of the footmark on the fence immediately below the reference mark was observed and recorded. The film was then advanced one frame and the process repeated, using the same car reference point. If the car advanced to a point where the first reference mark was obscured by the bushes, another car reference mark was selected and this frame was recorded as zero. The process continued until the car stopped.

Since the horizontal perpendicular distance from the camera to the reference fence and the distance from the reference fence to the car's tracks were known, the actual distance the car traveled in relation to that shown by the film was computed by a simple proportion. This corrected distance traveled per frame was then converted to miles per hour. Finally, all distances and time were equated to the front bumper of the test car and the data were used to plot speed-distance and speed-time curves for each test run.

For example, the speed-distance curve for test No. 1 is shown in figure 6, and the speed-time curve for the same test is shown in figure 7. Distances shown in figure 6 are the actual distances on the traveled path of the test car. In figure 7 the abscissa indicates the observed frame numbers, where one frame equals one forty-eighth of a second. Each plotted point is an average of five observations. All observations are in reference to the front bumper of the test car.

It is seen in figures 6 and 7 that the test car approached the contact point at 30.3 miles per hour. The film evaluation was the primary means used for determining the contact speed. From a study of the two curves, it is

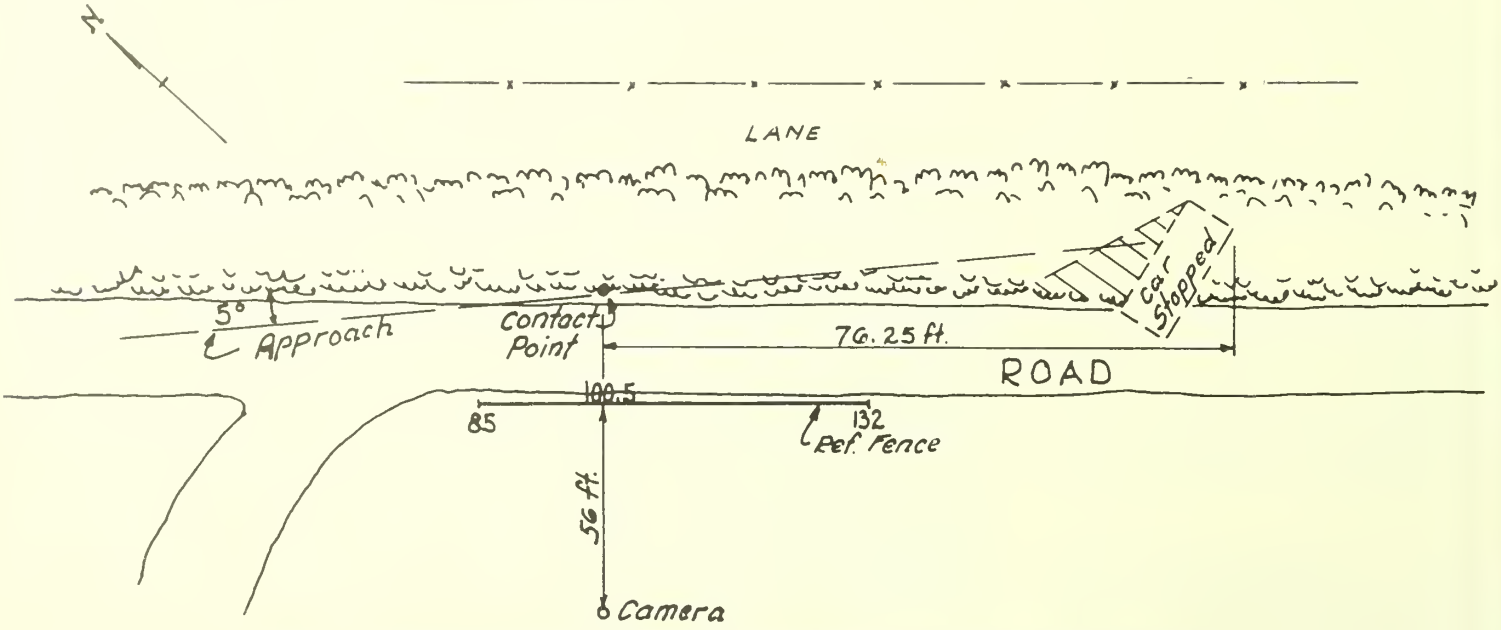

Figure 5.—Sketch of test No. 4 (5° angle approach at 30.2 miles per hour contact speed).

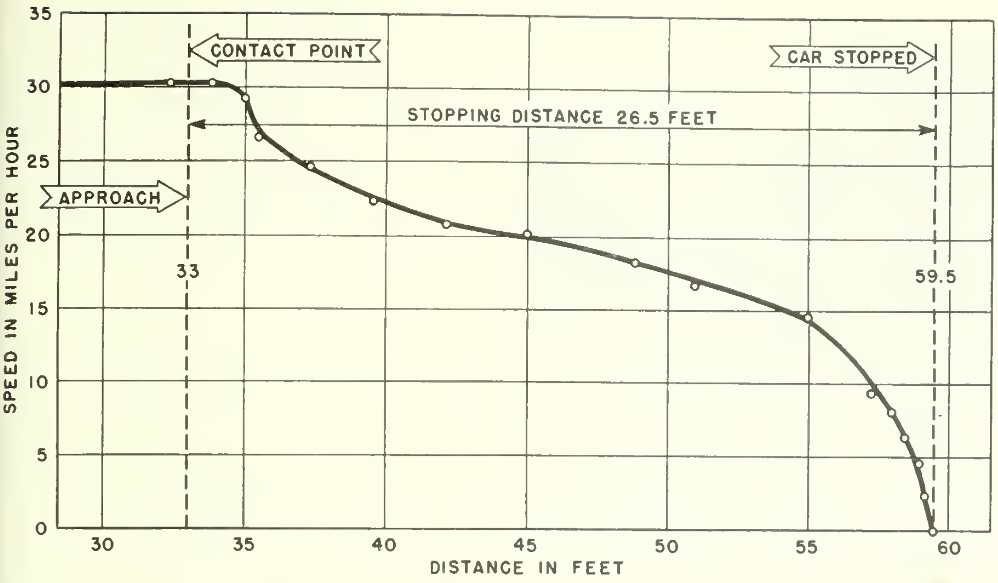

Figure 6.—Speed-distance curve for test No. 1.

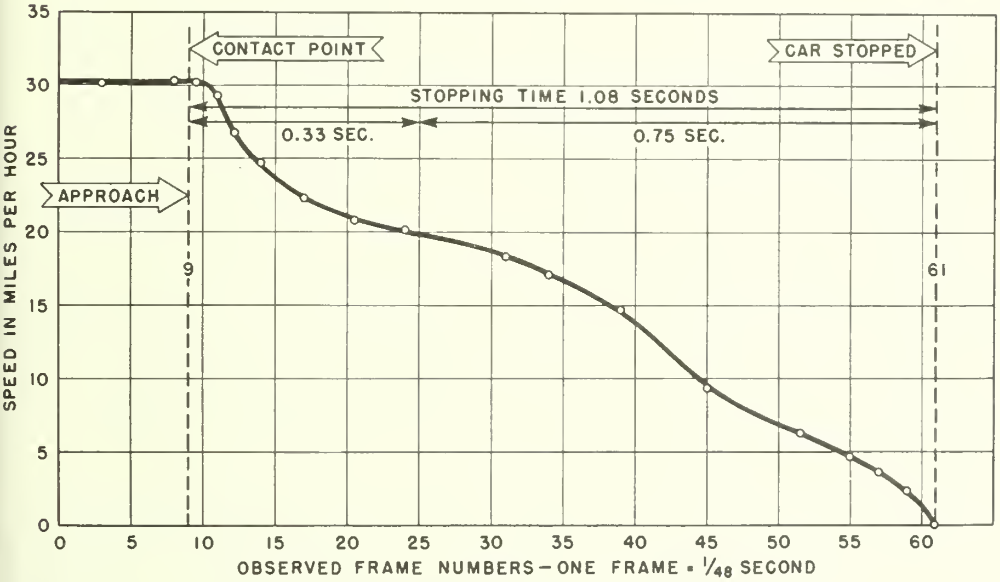

Figure 7.—Speed-time curve for test No. 1.

apparent that upon contact with the bushes, the car speed decreased rather rapidly to about 20 miles per hour in a distance of 12 feet and in a time interval of 0.33 second. The average deceleration for the period was 22.4 feet per second per second or 0.69 g. The car speed then dropped from 20 miles per hour to zero in a time interval of 0.75 second, which is equivalent to an average deceleration of 19.6 feet per second per second or 0.61 g. The overall stopping time was 1.08 seconds.

The maximum error in reading the distance traveled in one frame was generally observed to be plus or minus 0.1 foot. For the 48 fps film this amounted to plus or minus 3.27 m.p.h. Where the 128 fps film was used for evalutation, an error in distance observation of 0.1 foot produced a possible error in speed of plus or minus 8 m.p.h. However, the maximum deviation from the mean of the computed speeds, using the 128 fps film, was 6 m.p.h.

In several angle tests, the stopping distance was too great to be recorded by the fixed cameras. In other instances the car path curved sharply near the end of the run, which prevented complete evaluation of the car performance from the film record.

The evaluation methods and procedures were the same for the summer and winter tests. During the winter series the temperature was so low that all cameras seemed to be running slow. Later film evaluation proved the foregoing to be true, consequently the film results were not used. Stopping time for the winter tests was obtained solely from the accelerometer records.

Accelerometer-Tape Evaluation

The wax-coated accelerometer tapes for each test were photographed and the prints were projected and traced on cross-section paper. The enlarged graph was useful in determining the several peak deceleration values in relation to the time of occurrence and the average deceleration for the duration of the tests.

An enlarged graph of the longitudinal section of the chart for test No. 1 is shown in figure 8. Although this graph is not necessarily typical for all tests, it does indicate the type of data obtained from similar graphs prepared for all succeeding tests. It is seen that the maximum peak was 1.77 g. The elapsed time from the crash point for any peak and the total elapsed stopping time can be read from this graph. In this instance the maximum peak occurred about 0.34 second from the contact point. The overall stopping time was about 1.08 seconds. This stopping time is almost identical with that determined from the film evaluation of the same test, which was the case for all tests.

The average deceleration in the longitudinal plane was obtained by measuring the area under the curve with a planimeter and dividing the area by the base-line length between the point of contact and the point where the car came to rest. The average deceleration was determined for all tests in the longitudinal direction by this method and was used as a source of comparison of performance of the car and the bushes. In figure 8, it is indicated to be 1.05 g.

The evaluation of the tape for the single component accelerometer was read directly and the peak results compared with the vertical values obtained from the graph of the three-component accelerometer. In general, the single component accelerometer maximum range was inadequate, since the peak vertical acceleration was usually greater than 1 g. When the peak vertical acceleration was less than 1 g, the values of both instruments were in substantial agreement.

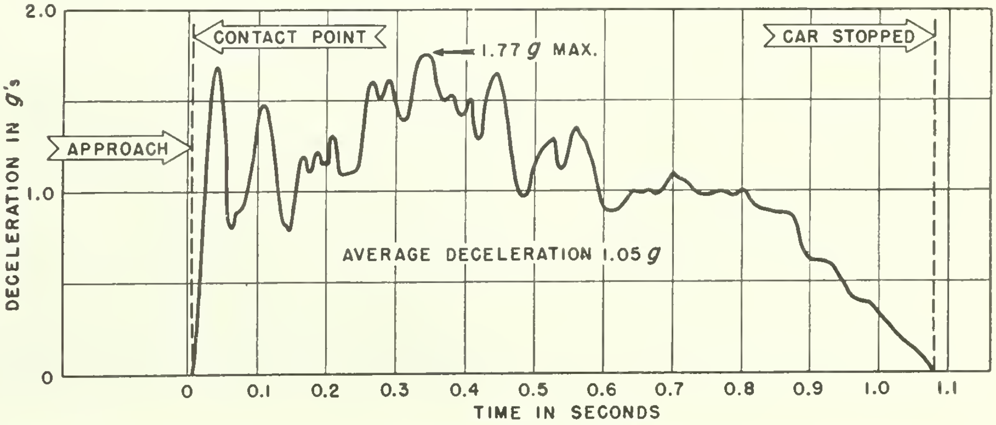

Figure 8.—Longitudinal deceleration during test No. 1, expressed in terms of gravity (g).

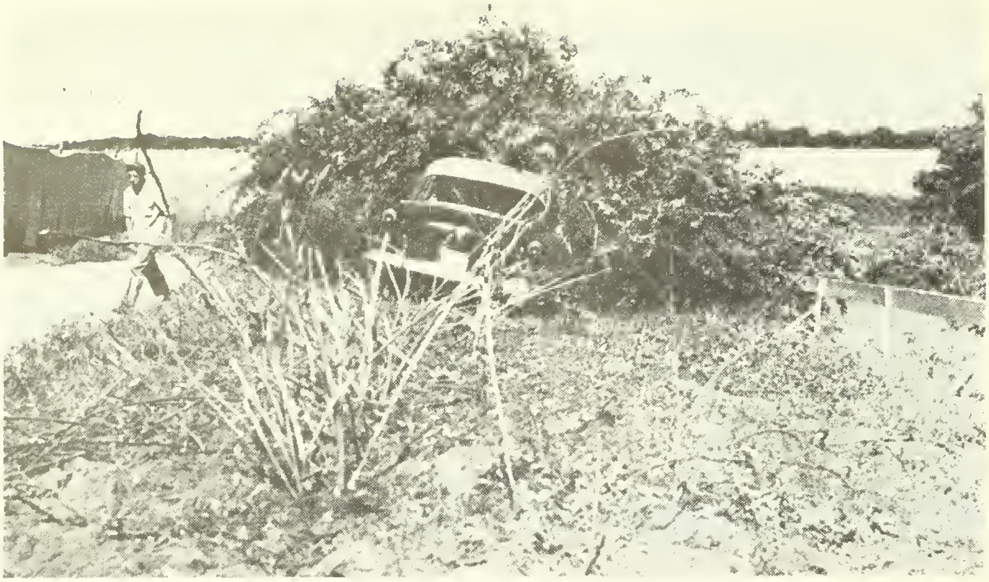

Figure 9.—Final position of test car after test No. 6.

Typical Performance Characteristics

During the first part of the deceleration period for the head-on tests, the canes of the bushes sheared about 3 to 4 feet above ground. Some of the bushes were pulled out and these plus the sheared canes and broken branches were pushed ahead of the car. The results of these actions are shown on the cover picture and in figure 9 for test No. 6. In this test the contact point was at 12 on the reference fence.

As the loose mass was accumulated and as the car pushed the entangled mass ahead, the forward bushes were either pushed down or were stripped of their foliage, as shown in figure 9. In the last one-third of the stopping interval, the loose mass was compressed with little forward movement, which resulted in the car being stopped very gradually. A tendency for the car to "ride up" on the bushes and the softness of the ground under the bushes make it doubtful that the use of brakes would have materially shortened the stop. In two tests where the hedge was wider than 8 feet, and where smaller seedling plants grew outside the axis, the car cut a path along the axis, leaving a fringe of bushes on each side, as shown in figure 10 for test No. 7.

Although the hedge was devoid of leaves in the winter tests, there appeared to be little, if any, significant difference in the stopping performance of the hedge between the summer and winter tests. With the ground frozen, there did appear to be more roots left in the ground.

Two angle tests at 5 degrees, one at 10 degrees, and one at 20 degrees were performed in the summer series. The speed of contact for these tests was planned for a maximum of 30 m.p.h. because the hedge was narrow and not too dense. In all angle tests, the fringe of the bushes did not slow the car appreciably; it was not until the front of the car encountered the central mass of the hedge that the vehicle was noticeably retarded. Since relatively few bushes were in the path of the car, they were either sheared or torn out. There was no massive accumulation of loose bushes to compress, as described previously. Had there been several parallel rows of bushes instead of one, the performance would have been similar to a head-on crash once the vehicle was turned into the hedge.

The car was stopped within the hedge on both 5- and 10-degree angle tests and passed through the hedge on the 20-degree angle test. There was no indication that the bushes tended to deflect the car. Actually, the retarding effect of the bushes caused the car to swing inward on an arc, as shown in figure 5.

During the summer and winter tests, there were three different drivers employed. One common observation was of considerable significance; namely, that once the car entered the hedge the driver had no further control of the steering. When the approach of the car was on the line of the axis of the bushes, the car path throughout the test was generally straight. However, toward the end of the stopping period the rear of the car frequently moved laterally as the mass of loose hedge became thoroughly compressed.

Test Results

A summary of the test data is given in table 1 for the 12 tests. A study of the data obtained from any one test, or a comparison of the results of similar tests, must be made with caution because there were several factors affecting the results that did not remain constant for each test. Three of the more important factors were: The width, height, and compactness or density of the bushes; the pattern of the path the test car followed on the approach, and during the stopping period; and the condition of the ground surface on the approach and within the hedge. In order to help evaluate the test results contained in table 1, a brief discussion of each test follows.

Summer tests, head-on approach

Test No. 1 was made as a pilot run at low speed to study the performance and to develop procedure. The approach to the axis of the hedge required that the driver execute a reverse curve maneuver of about 9 feet laterally in 60 feet of distance to crash the hedge parallel to the hedge axis. The car tracks indicated that the approach was not parallel to and on the hedge axis; consequently, the rear of the car moved gradually sidewise about 3 feet toward the end of the stopping period. The front of the vehicle remained on the axis.

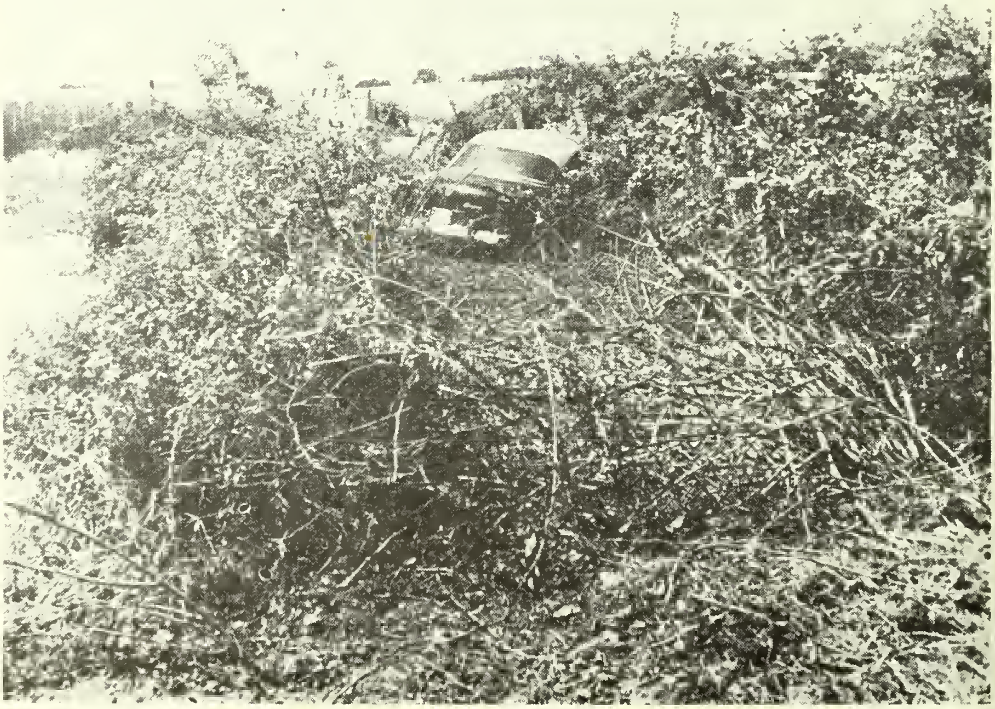

Figure 10.—Final position of test car after test No. 7.

Table 1.—Summary of crash test results [1]

Test No.	Collision course	Contact speed	Stopping distance	Stopping time	Longitudinal deceleration [2]			Maximum acceleration [2]	
					Average	Maximum	Time from contact	Vertical	Lateral
		M.p.h.	*Ft.*	*Sec.*	*g*	*g*	*Sec.*	*g*	*g*
1	Head on	30.3	26.5	1.08	1.05	1.77	0.33	1.25	1.0
2	do	30.5	26.0	1.25	.5	----	----	.7	.25
3	do	21.0	17.0	1.02	----	----	----	----	----
4	5° angle	30.2	76.2	2.8	----	.87	1.9	.5	.5
5	do	36.0	117	1.8	.75	1.40	.69	1.5	1.25
6	Head on	35.5	51.8	1.91	.68	2.72	.21	1.0	.5
7	do	47.8	57.2	1.94	.93	2.24	.38	1.39	2.12
8	20° angle	28.4	----	----	----	1.09	.13	1.33	.7
9	10° angle	27.0	56+	1.62	----	1.71	1.00	1.12	.91
10	Head on	35.0	50.1	1.50	----	----	----	----	----
11	do	22.0	16.3	1.26	.53	.82	.25	.5	.5
12	do	50+	76.5	2.14	.64	1.74	.32	1.23	1.27

[1] Tests 1-9 were conducted during the summer, and tests 10-12, during the winter.
[2] Decelerations and accelerations were recorded by three-component accelerometer.

This sidewise movement probably would have been contained had the hedge been wider.

Most of the bushes on the car path were pulled out because the soil was dry and loose. Some bushes were sheared. The measured stopping distance of 26.5 feet and other data are given in table 1. The entire hood of the car up to the windshield was buried in a compressed mass of tangled rose bushes, but the car was removed under its own power. An area about 20 feet in length was swept clean of bushes.

Test No. 2 was a low-speed test intended to duplicate test No. 1. Unfortunately the car struck to the right of the hedge center, causing the rear end to slide severely to the right. The sliding action which developed rendered the test unusable for comparative purposes with respect to the accelerometer data. The stopping distance of 26.0 feet closely approximated that for test No. 1. The final position of the car is shown in figure 11.

Test No. 3 was planned to replace test No. 2, using a straight approach over adjacent pastureland. The available distance proved to be insufficient to develop the planned approach speed; however, the rear end did not slide to the right. This test definitely proved that the angle of approach coupled with the loose soil was causing the sliding action in previous tests. The accelerometers were not employed on this test which accounts for the absence of such data in table 1.

Summer tests, angle approach

Tests Nos. 4 and 5 were conducted as 5-degree angle tests for duplicate study. In both tests the stopping distances were excessive because the distances were measured from the point where the car first touched the fringe of overhanging bushes. The path of the car was straight until the central bush mass was encountered; thereafter the path curved into the central hedge structure.

In test No. 4, the car was stopped by the hedge and remained within the hedge mass, as illustrated in figure 5, even though the bushes were thin and less dense than at other locations. In test No. 5, the rear of the car swung rather violently in an arc when it struck a large central root mass. It stopped perpendicular to the axis of the hedge 117 feet beyond the contact point. Had the approach speed been higher, the tests would have been extremely hazardous because of fixed objects located on the far side of the hedge.

Summer tests, head-on approach

Test No. 6 was planned for a contact speed of 50 m.p.h.; however, the approach surface was soft and irregular even after preparation, and the approach speed decreased rapidly to 35.5 m.p.h. at contact. The hedge was wide and dense. This test demonstrated most effectively the desirable properties of the multiflora rose hedge as a crash barrier.

The highest deceleration, 2.72 *g*, occurred in this test. It is significant to note that 0.21 second was required to develop this peak value from the instant of impact. The gradual buildup to the peak deceleration is an important advantage demonstrated by this and other tests.

In this particular test the individual bushes were closer than usual, evenly spaced, and the canes were large, as shown in figure 9. The average diameter was about three-quarters of an inch and the maximum was $1\frac{3}{8}$ inches. The bushes were counted before and after the test and there were 26 bushes destroyed in the test. When the enlarged longitudinal section of the accelerometer record was analyzed, there were 26 distinct peaks averaging 1.25 *g* over about two-thirds of the stopping distance of 51.8 feet. Field notes indicated that the bushes were sheared completely in the first half of the stopping distance.

These observations indicate that each bush offered a distinct and nearly equal resistance in overcoming the kinetic energy of the car, which was finally reduced to zero by the compression of the loose entangled mass. A study of the speed-distance and speed-time curves indicated that the compression of the mass started at about 47 feet or 0.92 second from the point of contact.

Test No. 7 was similar to test No. 6 except that the contact speed was 47.8 m.p.h. In order that the effect of a discontinuous hedge be studied, 10 feet of hedge were removed for the full width at a point 55 feet ahead of the contact point. This distance was selected on the basis of the stopping distance observed in previous tests and on the characteristics of the hedge, which was wider and more dense than any section previously tested.

The car was declutched about 15 feet ahead of the contact point. It followed a straight path along the axis of the bushes until near the end of the run, when the rear end moved laterally until it came to rest at an angle of about 45 degrees with the axis. The final position of the car is shown in figure 10 as well as the path made through the bushes. About 3 to 4 feet of partially damaged bushes can be seen on both sides. The mass of bushes above the hood of the car had not yet settled at the time of the picture.

The 10-foot gap, previously described, was partially filled. It was found that the compressed mass was pushed into the opening only 6 feet on the center position and that no intrusion occurred on either edge of the hedge. Since the gap was only partially filled with the compressed mass, it may be concluded that the unsheared canes ahead of the car prevented the mass from being pushed forward, consequently the mattress was pushed upward as it was being compressed. It was

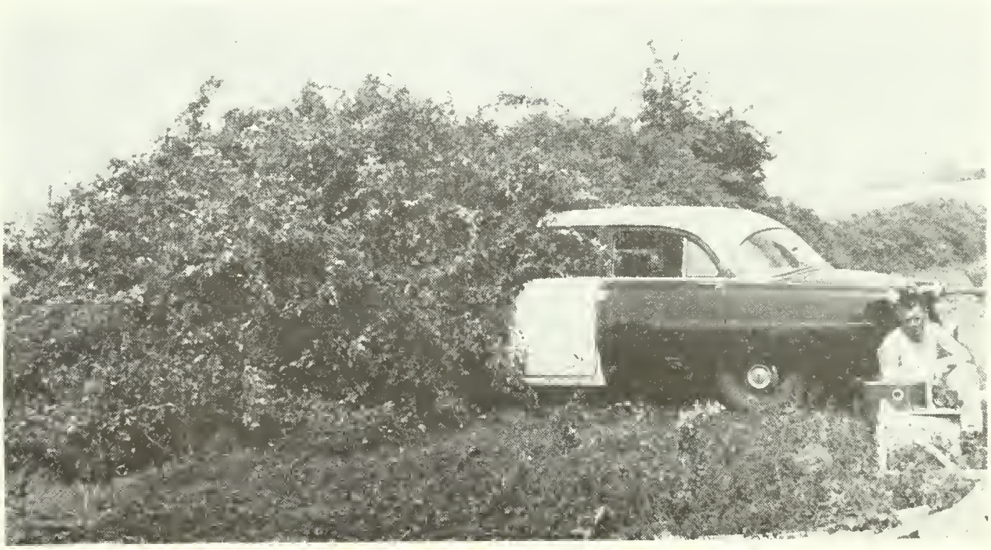

Figure 11.—Final position of test car after test No. 2.

generally noted that the front of the car usually "rode up on" the mass in the last short interval of the deceleration period and that the front end settled slowly after the car stopped.

A study of a photographic enlargement of the accelerometer record indicated a stopping time of 2.03 seconds, which was in fair agreement with the 1.94 seconds obtained by film analysis. This study also revealed several peak values in longitudinal deceleration, the maximum being 2.24 *g* at 0.38 second after contact. The longitudinal deceleration curve for this test was not so erratic as that of test No. 6. It appears from the graph that the last 0.8 second was consumed in compressing the loose mass of bushes during which time very little shearing action occurred.

As noted in table 1, this test produced the maximum peak values of vertical and lateral acceleration. The vertical *g* factor was 1.39, occurring 0.46 second after contact. There were other vertical peak values spread over the graph which had values of 1.0 to 1.14 *g*. These observations are in agreement with the car performance which showed a violent pitching action in the longitudinal plane. On the lateral acceleration record a severe maximum *g* factor of 2.12 occurred 0.93 second after contact. A comparison of the enlarged accelerometer record in each plane indicated a rapid and violent lateral movement or skidding of the rear end of the test car. However, the driver did not report any serious acceleration reaction from the seat or shoulder belts.

Summer tests, angle approach

Test No. 8 was a 20-degree angle test. The contact speed was planned for 30 m.p.h. because it was expected that the car would swerve through the thin 8-foot-wide hedge. The car passed completely through as predicted and the brakes were applied outside the hedge to prevent serious injury. As a result, little reliable data for evaluation were available.

The car followed a straight line for about 20 feet, after which it veered left on an arc. It was finally stopped 43 feet from the contact point measured along the axis of the hedge. Figure 12 shows the vehicle after it had crashed through and was stopped with the help of its brakes.

Test No. 9 was run on a section of the hedge shown in the foreground of figure 1. It was made on a 10-degree angle approach and was planned for a speed of 30 m.p.h. The performance of the car was similar to that for previous angle tests. The car was stopped by the hedge, but the impact with the central mass caused the rear of the car to swing violently through an arc of about 80 degrees to the axis. Had the hedge at this point been wider and more dense, a more gradual stop would probably have occurred.

Winter tests, head-on approach

Test No. 10 was planned to duplicate the speed obtained in test No. 6. It is noted in table 1 that the stopping distance and elapsed time for these tests were quite similar. The hedge was similar in width and density although the ground within the hedge on test No. 10 was quite irregular and many rocks were partly exposed. The approach was on a slight downgrade and there was a considerable cross slope to the ground. Although the bushes were devoid of leaves, the performance of the hedge was similar in all respects to that described for test No. 6.

Test No. 11 was planned to duplicate the summer test No. 3, which had a contact speed of 21 m.p.h. Because the remaining hedge in this section was located on very rough and rock-strewn ground, it was too dangerous to run high-speed tests which would have required long stopping distances. The results of tests Nos. 3 and 11, as indicated in table 1, were quite similar.

Test No. 12 was planned for a contact speed of 50 m.p.h. on the remaining section of hedge suitable for testing. This test was run on the far end of the hedge shown in figure 1. The hedge was thin but measured 9 feet wide and 108 feet long. There were 36 individual bushes in this length. The stopping distance, when compared to that of test No. 7, was

(Continued on page 267)

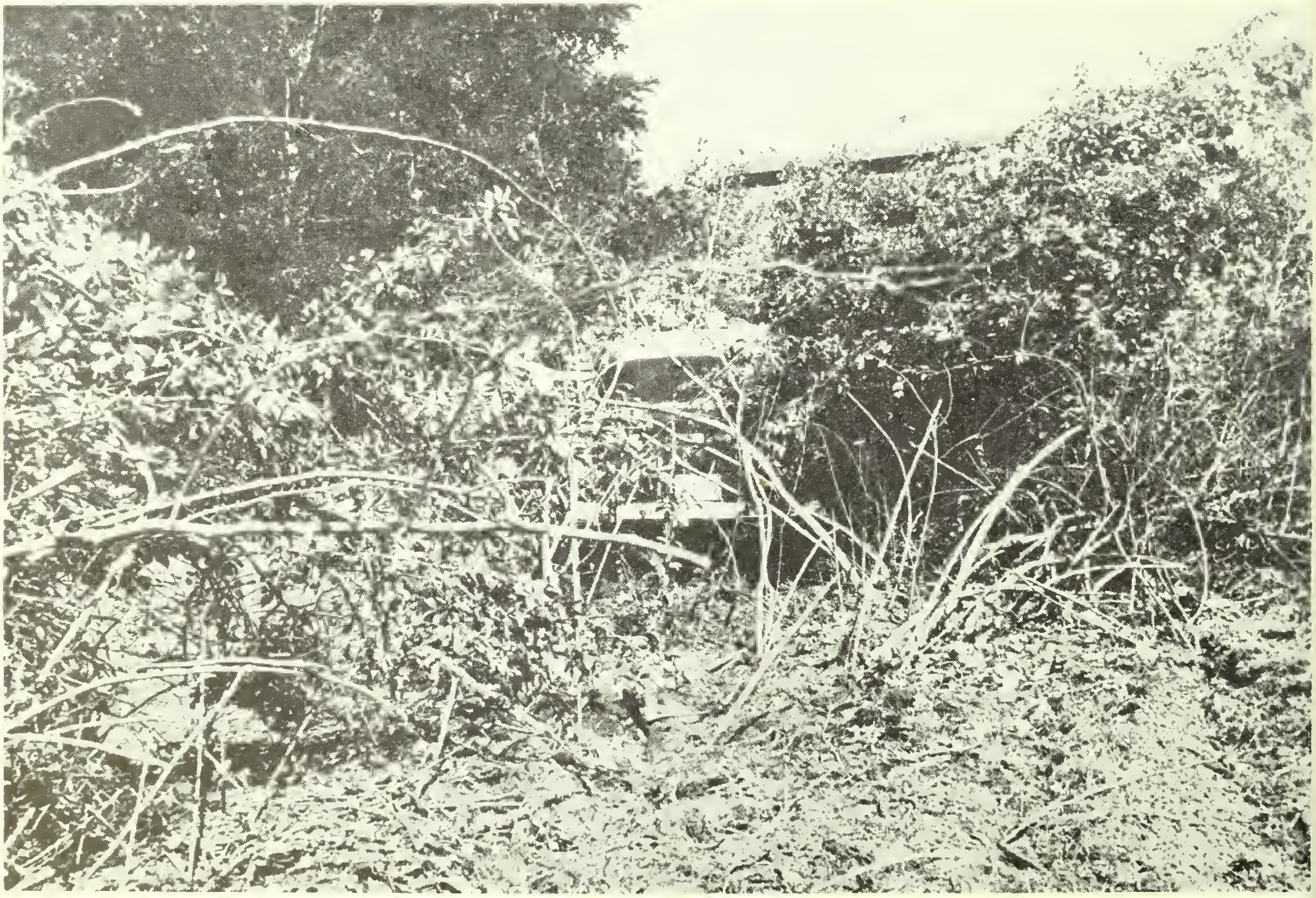

Figure 12.—Final position of test car after test No. 8. (Vehicle crashed through the hedge and was stopped with the aid of brakes.)

Traffic and Travel Trends, 1956

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH BUREAU OF PUBLIC ROADS

Reported by THOMAS B. DIMMICK,
Supervising Transportation Economist

Traffic studies undertaken by the Bureau in cooperation with the State highway departments are a continuing operation, and the vast amount of data resulting from these studies provides interested officials, both public and private, with an overall view of the use of the highways.

The trends reported in the annual series of articles relating to travel by passeger cars, trucks, and buses, volumes of freight transported over the highways, and frequencies of heavy gross loads and axle loads are of prime importance to agencies responsible for providing highway facilities.

In the last 5 years, 1951–56, travel on all roads and streets has increased 28 percent, with the greatest increase, 31 percent, occurring in rural areas as compared with 24 percent in urban areas. During this period, the growth of passenger-car travel exceeded all other types of vehicles as evidenced by increases of 29, 22, and 12 percent for passenger cars, trucks, and buses, respectively.

A considerable portion of this article is devoted to travel and loading practices observed on the main rural roads, which comprise about 12 percent of all rural mileage and carry over 67 percent of all rural traffic. Approximately two-thirds of all truck travel in rural areas was performed on these roads, of which about one-third was by truck combinations.

Average daily travel on the main rural roads totaled 650 million vehicle-miles in 1956, as compared with 625 million in 1955, a 4-percent increase. By principal geographical areas, the rates of increase in travel from 1955 to 1956 were 5.3 percent for the States in the eastern region, 2.6 percent for States in the central region, and 5.6 percent for States in the western region.

Passenger-car and bus travel on main rural roads increased 26 percent between 1951 and 1956; single-unit truck travel increased 22 percent; and travel by truck combinations increased 27 percent.

In 1956, 57 percent of all freight-carrying vehicles were loaded, and weighed an average of about 24,200 pounds. For the period 1951-56, weights of loaded single-unit trucks decreased over 3 percent, whereas those of loaded combinations increased about 6 percent.

Single-unit trucks in 1956 carried loads during 51 percent of their travel as compared with 60-65 percent during the prewar period, 1936-41. Combinations in 1956 were found to be loaded during 70 percent of their travel as compared with 72 percent in 1936.

Average loads carried by single-unit trucks increased from 1.86 tons in 1936 to 2.39 tons in 1956, a 28-percent increase, while combinations increased from 6.90 tons in 1936 to 11.51 tons in 1956, a 67-percent increase.

The volume of freight hauled in 1956 by single-unit trucks was 41.2 billion ton-miles as compared with 14.3 billion in 1936; combinations in 1956 hauled 130 billion ton-miles as compared with 13.7 billion in 1936. The 2-axle, 6-tire trucks, the principal load-carrying single-unit trucks, accounted for 25 percent of all truck travel in 1956, and less than 16 percent of the ton-mileage; truck-tractor and semitrailer combinations accounted for 30 percent of the travel and 67 percent of the ton-mileage.

Frequencies of freight-carrying vehicles weighing 30,000, 40,000, and 50,000 pounds or more reached a new high in 1956. Since 1936 the number of trucks in each 1,000 loaded and empty vehicles weighing 30,000 pounds or more have increased almost 5 times; for 40,000 pounds or more, over 13 times; and 50,000 pounds or more, 31 times. From 1951 to 1956, the frequencies increased 11, 24, and 45 percent, respectively.

Frequencies of axles weighing 18,000, 20,000, and 22,000 pounds or more show increases in 1956 over 1954 and 1955 but for the period 1951-56, there has been a decrease of 8, 15, and 14 percent in the three respective axle-weight categories.

THIS article, discussing the significance of annual changes and trends in highway travel, is a continuation of a series that have been published each year since 1946. A comprehensive study of automobile and truck usage of the highways is presently underway. One of the provisions of the Federal-Aid Highway Act of 1956, particularly section 210 of the Highway Revenue Act, requires, among other stipulations, that the Bureau of Public Roads inquire into the effects on design, construction, and maintenance of Federal-aid highways of the magnitude and frequency of occurrence in the traffic stream of vehicles of different sizes and weights. At this time, only limited data from this study are available for inclusion in this article, but a similar article to be published in 1958 will be based on much more complete information concerning highway usage.

In the 5-year period beginning in 1936, 47 of the 48 States in cooperation with the Bureau of Public Roads conducted surveys for a 12-month period to collect data which would supply comprehensive information concerning vehicle characteristics and travel habits. The measuring of road mileages, the counting of traffic by vehicle types, the weighing of trucks on rural roads, and the questioning of drivers concerning origin and destination and miles driven on different road systems during the preceding year supplied basic data from which a vast amount of information regarding travel habits, ton-miles of freight carried on rural systems, and vehicle-miles driven on all systems could be calculated for the period of the survey.

Since the original surveys were made, the States have used automatic traffic recorders continuously at a large number of locations and have adopted other continuing operations which provide data for estimating trends in traffic volumes. Periodic weighing operations, combined with manual classification counts of all vehicles passing the weighing stations, have been made which provide information concerning vehicle types and weights as well as their loading. By means of these trends annual estimates have been published showing for each year the travel on rural roads and city streets from 1936 to 1955.[1] By combining carried load with vehicle-mileage data on main rural roads, the ton-mileage of freight hauled on these highways has been estimated for each year. Sufficient data are not now available to justify publishing estimates of ton-mileage on local roads. No data have been collected in past surveys concerning loads carried on city streets, although the survey now underway should provide such information.

Travel Related to Economic Trend

The trends in travel on all rural roads and streets, motor-vehicle registrations, motor-fuel consumption, and gross national product (in 1947 dollars) are shown in figure 1 for the years 1936–56, inclusive, as a percentage of the 1950 totals. This chart indicates that with the exception of the war years and the early postwar period, the trend of travel and its related factors of registration and fuel consumption followed closely the economic trend, as represented by the gross national product, until 1954. In 1954 the gross national product shows a downtrend, the total adjusted to the 1947 dollar decreasing from $306.6 billion in

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[1] See previous articles on traffic in PUBLIC ROADS: Vol. 29, No. 5; vol. 28, No. 11; vol. 27, Nos. 6 and 11; vol. 26, Nos. 5 and 11; vol. 25, Nos. 3, 7, and 12; vol. 24, No. 10; and vol. 23, No. 9.

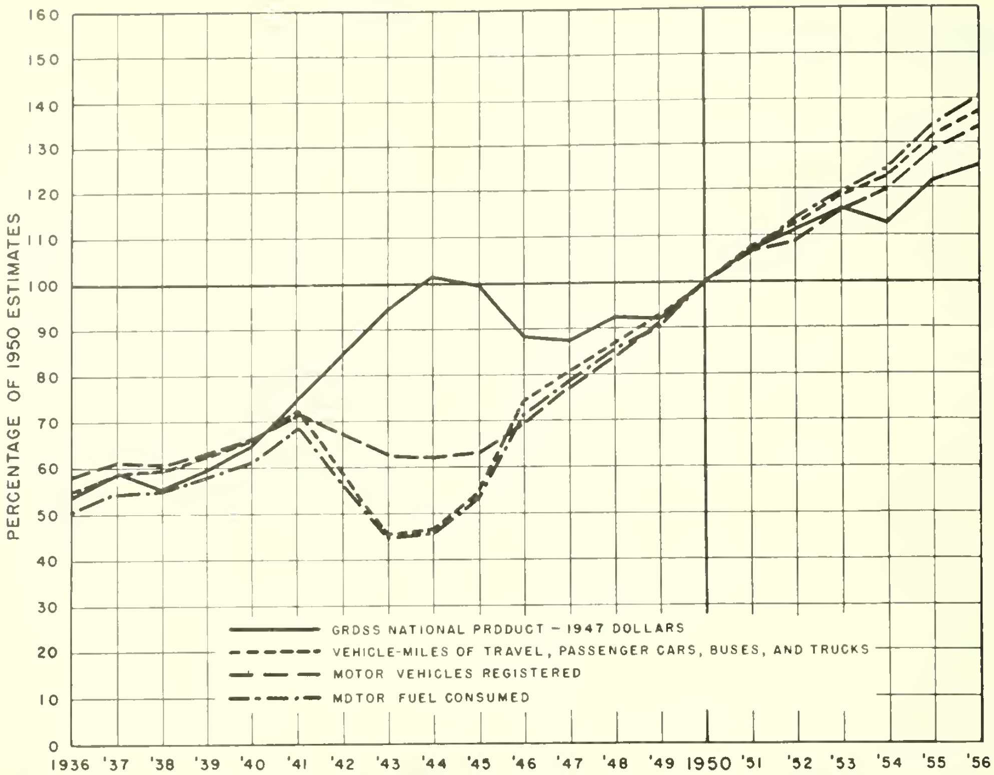

Figure 1.—Total travel, motor-vehicle registration, motor-fuel consumption, and gross national product, 1936–56, as a percentage of the respective amounts in 1950.

1953 to $300.8 billion in 1954, or 1.9 percent. At the same time, total travel increased 3.0 percent while registrations and fuel consumption both increased 3.7 percent.

From 1954 to 1955, the adjusted gross national product increased 7.2 percent; total travel increased 7.6 percent; registrations, 7.5 percent; and fuel consumption, 7.7 percent. Thus all of the trends were again about parallel. The 1956 figures indicate that all of the trends for the indices considered were again leveling off. Increases in 1956 were only 2.5 percent for the adjusted gross national product, 4.1 percent for total travel, 4.0 percent for vehicle registrations, and 5.2 percent for motor-fuel consumption. Thus once again as in 1954, the index of the national economy was not fully reflected in the indices of highway usage.

A comparison of vehicle-miles of travel on all roads and streets by 5-year periods commencing in 1936 is given in table 1. Probably the most significant relation shown by this table is the greater increase of travel in most periods by trucks and truck combinations in comparison with passenger cars and buses. Travel by trucks and truck combinations in 1956 was 182 percent greater than in 1936, while during that period passenger-car travel increased 143 percent, bus travel increased 95 percent, and total travel increased 149 percent. Trucks and truck combinations accounted for 16.3 percent of all rural and urban travel in 1936, and 18.5 percent in 1956.

In contrast with the overall trends of travel for the 20-year period, 1936–56, the 1951 and 1956 vehicle-mileage figures show that in the last 5-year period the rates of growth of passenger-car travel and of truck and truck-combination travel were reversed from that found in the third 5-year period (1946–51). For instance, passenger-car travel in 1956 was 29 percent higher than in 1951, while travel by trucks and truck combinations was only 22 percent higher; the rates of increase from 1946 to 1951 were 40 percent for passenger cars and 69 percent for trucks and truck combinations.

Travel on All Roads and Streets

Table 2 shows the estimated travel in 1956 on main rural roads, local rural roads, and urban streets for passenger cars, buses, and trucks, together with the number of vehicles registered and quantity of fuel consumed on the highways. The travel figures were obtained mainly by applying trends obtained in 1956 from the automatic traffic recorders to the 1953 data which were derived from the various State reports submitted for the nationwide highway study of that year. Actual vehicle-mileage totals on the various turnpikes were obtained from reports of the turnpike authorities. Finally, minor adjustments were made in the totals of local rural road and city street travel to bring the total travel figure for the year to the total of the latest estimates submitted by the States and the District of Columbia. Such adjustments are necessary periodically to add the small but important increment of travel on improved or newly constructed sections of local roads and streets in expanding fringe areas where traffic counts ordinarily are not made.

Urban travel comprised about 44 percent of the total 1956 travel. Since 1951 the relation between urban and rural travel has remained nearly constant, but in the earlier years a different division existed. In 1946, urban travel constituted almost 50 percent of the total travel; 49 percent in 1941; and 51 percent in 1936. These data show that in the period immediately previous to 1951, the volume of rural travel increased at a more rapid rate than urban travel, while from 1951 through 1956 the two rates of increase have been approximately equal.

The concentration of truck travel on main rural roads also may be noted in table 2. These main roads, which comprise about 12 percent of the mileage of all roads and streets, carry over 43 percent of all truck travel compared to 37 percent for passenger cars. As would be expected, the larger portion of the heavier vehicles use the main highways. Approximately 32 percent of the truck traffic on these facilities was made up of combination-type vehicles, but on the local roads the combinations accounted for only 10 percent of the truck traffic. To illustrate further the relation of truck travel on the two classes of rural roads, the average daily number of trucks traveling at any point on the main rural highways in 1956 was 238 single-unit trucks and 114 truck combinations; on local rural roads, the averages were 22 single-unit trucks and only 3 combinations. Sufficient data concerning the vehicle types using city streets are not yet available to make a similar distribution possible in urban areas.

Table 1.—Comparison of estimated vehicle-miles of travel on all roads and streets in 1936, 1941, 1946, 1951, and 1956

Year	Total travel, vehicle-miles	Passenger-car travel		Bus travel		Truck and truck-combination travel	
		Vehicle-miles	Percentage of total travel	Vehicle-miles	Percentage of total travel	Vehicle-miles	Percentage of total travel
	Millions	*Millions*		*Millions*		*Millions*	
1936	252,128	208,654	82.76	2,367	0.94	41,107	16.30
1941	333,612	275,839	82.68	2,820	.85	54,953	16.47
1941:1936 ratio	*1.32*	*1.32*	*1.00*	*1.19*	*.90*	*1.34*	*1.01*
1946	340,880	280.597	82.31	4,053	1.19	56.230	16.50
1946:1941 ratio	*1.02*	*1.02*	*1.00*	*1.44*	*1.40*	*1.02*	*1.00*
1946:1936 ratio	*1.35*	*1.34*	*.99*	*1.71*	*1.27*	*1.37*	*1.01*
1951	491,093	392.131	79.85	4,118	.84	94,844	19.31
1951:1946 ratio	*1.44*	*1.40*	*.97*	*1.02*	*.71*	*1.69*	*1.17*
1951:1941 ratio	*1.47*	*1.42*	*.97*	*1.46*	*.99*	*1.73*	*1.17*
1951:1936 ratio	*1.95*	*1.88*	*.96*	*1.74*	*.89*	*2.31*	*1.18*
1956	627,843	507,138	80.78	4,605	.73	116,100	18.49
1956:1951 ratio	*1.28*	*1.29*	*1.01*	*1.12*	*.87*	*1.22*	*.96*
1956:1946 ratio	*1.84*	*1.81*	*.98*	*1.14*	*.61*	*2.06*	*1.12*
1956:1941 ratio	*1.88*	*1.84*	*.98*	*1.63*	*.86*	*2.11*	*1.12*
1956:1936 ratio	*2.49*	*2.43*	*.98*	*1.95*	*.78*	*2.82*	*1.13*

Table 2.—Estimate of motor-vehicle travel in the United States by vehicle types in the calendar year 1956

Vehicle type	Motor-vehicle travel					Number of registered vehicles [1]	Average travel per vehicle	Motor-fuel consumption		Average travel per gallon of fuel consumed
	Main rural road travel	Local rural road travel	Total rural travel	Urban travel	Total travel			Total [2]	Average per vehicle	
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles/gal.*
Passenger cars [3]	185, 901	89, 785	275, 686	231, 452	507, 138	54, 249	9, 348	35, 326	651	14. 36
Buses:										
Commercial	1, 114	303	1, 417	1, 854	3, 271	95	34, 432	673	7, 084	4. 86
School and nonrevenue	663	521	1, 184	150	1, 334	160	8, 338	129	806	10. 34
All buses	1, 777	824	2, 601	2, 004	4, 605	255	18, 059	802	3, 145	5. 74
All passenger vehicles	187, 678	90, 609	278, 287	233, 456	511, 743	54, 504	9, 389	36, 128	663	14. 16
Trucks and combinations	50, 070	24, 022	74, 092	42, 008	116, 100	10, 737	10, 813	13, 978	1, 302	8. 31
All vehicles	237, 748	114, 631	352, 379	275, 464	627, 843	65, 241	9, 623	50, 106	768	12. 53

[1] Registration figures differ slightly from those in Bureau of Public Roads table MV-1 because of adjustments in classification in a few States of lightweight farm trucks.

[2] Total fuel consumed differs from that given in Bureau of Public Roads table G-21 because of adjustments to cover estimated amounts used by motorcycles.

[3] Includes taxicabs.

Data have been collected concerning the loads carried on local roads, but such studies were limited in scope and therefore not as reliable as the information on main roads. Local road mileage far exceeds that of the main roads, yet estimates indicate that truck travel on main roads was more than double and ton-mileage was more than four times the amount carried on local roads. Because of the limited data and the relative unimportance of the local road mileage from a freight-carrying standpoint, discussion in subsequent sections of this article is confined to the main rural roads.

Main Rural Road Travel Increases

Figure 2 shows the annual vehicle-miles of travel by all vehicles on main rural roads by 12-month periods ending each month (moving average) from the end of 1936, the first year of the planning surveys, to August 1957. This method of presentation eliminates the seasonal fluctuations. From the end of 1946 through 1951, the increase of each year's traffic over the previous year averaged approximately 10 percent. Since 1951 the annual increases over the previous year have been somewhat smaller: 1952, 7 percent; 1953, 5 percent; 1954, 2 percent; 1955, 5 percent; 1956, 4 percent; and the first 8 months of 1957, 3 percent.

The average daily vehicle-miles of travel on main rural roads by months are shown in figure 3 for 1955, 1956, and the first 8 months of 1957. The graph shows that travel in 1956 was greater month by month for the United States as a whole than it was in the corresponding months of the previous year. Likewise for the first 8 months of 1957, travel exceeded that in 1956.

The average daily travel on main rural roads in 1956 totaled 650 million vehicle-miles as compared with 625 million vehicle-miles in the previous year, a 4.0-percent increase. A comparison of average daily travel by census regions [2] in 1956 and 1955

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[2] *Eastern region.*—New England division: Connecticut, Maine, Massachusetts, New Hampshire, Rhode Island, and Vermont. Middle Atlantic division: New Jersey, New York, and Pennsylvania. South Atlantic division: Delaware, Florida, Georgia, Maryland, North Carolina, South Carolina, Virginia, and West Virginia. *Central region.*—East North Central division: Illinois, Indiana, Michigan, Ohio, and Wisconsin. East South Central division: Alabama, Kentucky, Mississippi, and Tennessee. West North Central division: Iowa, Kansas, Minnesota, Missouri, Nebraska, North Dakota, and South Dakota. West South Central division: Arkansas, Louisiana, Oklahoma, and Texas. *Western region.*—Mountain division: Arizona, Colorado, Idaho, Montana, Nevada, New Mexico, Utah, and Wyoming. Pacific division: California, Oregon, and Washington.

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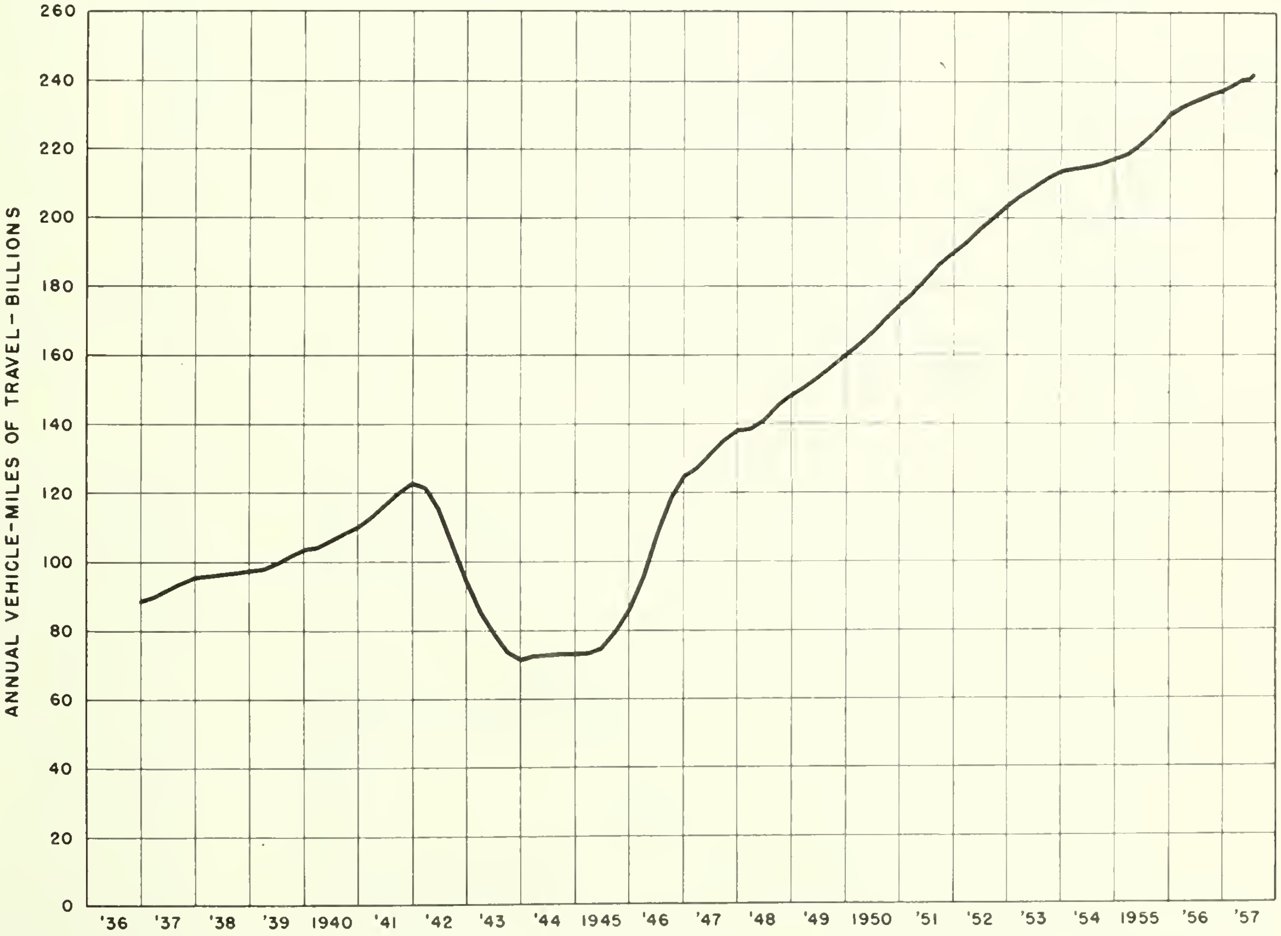


Figure 2.—Vehicle-miles of travel on main rural roads by 12-month periods ending each month, 1936 to August 1957.

STATE LEGAL MAXIMUM LIMITS OF MOTOR VEHIC

Prepared by the B

Line	State	Width inches[1]	Height ft.-in.	Length—feet[2]				Number of towed units[3]			Axle load—pounds				Ty
				Single unit		Truck tractor semitrailer	Other combination	Semitrailer	Full trailer	Semitrailer and full trailer	Single		Tandem		
				Truck	Bus						Statutory limit	Including statutory enforcement tolerance	Statutory limit	Including statutory enforcement tolerance	
1	Alabama	96	[6]12-6	35	40	50	NP	1	NP	NP	18,000	19,800	36,000	39,600	Tabl
2	Arizona	96	13-6	40	40	65	65	1	1	2	18,000		32,000		Tabl
3	Arkansas	96	13-6	35	40	50	50	1	1	NP	18,000	[7]18,500	32,000	32,500	Spec
4	California	96	13-6	35	[9]35	[10]60	60	NR	NR	NR	18,000		32,000		Tabl
5	Colorado	[11]96	13-6	35	40	60	60	1	2	2	18,000		36,000		Form
6	Connecticut	102	12-6	45	45	[12]45	NP	1	NP	NP	22,400	22,843	36,000	36,720	Spec
7	Delaware	96	[6]12-6	35	42	50	60	1	1	2	20,000		36,000		Tabl
8	District of Columbia	96	12-6	35	35	50	50	1	1	NP	22,000		38,000		Tabl
9	Florida	96	[6]12-6	[14]35	40	50	50	1	1	NP	20,000	22,000	40,000	44,000	Tabl
10	Georgia	96	13-6	[15]+39	[15]+45	48	48	1	1	NP	18,000	20,340	36,000	40,680	Spec
11	Hawaii	108	13-0	40	40	55	65	1	1	2	24,000		32,000		Form
12	Idaho	[18]96	14-0	35	[19]40	60	65	1	1	2	[20]18,000		[20]32,000		Tabl
13	Illinois	96	13-6	42	42	50	50	1	1	2	[21]18,000		32,000		Spec
14	Indiana	96	13-6	36	40	50	50	1	1	2	[23]18,000	[23]19,000	[23]32,000	[23]33,000	Spec
15	Iowa	96	[6]12-6	35	[19]40	50	NP	1	NP	NP	18,000	18,540	32,000	32,960	Tabl
16	Kansas	96	12-6	35	[19]40	50	50	1	1	NP	18,000		32,000		Tabl
17	Kentucky	96	[25]12-6	[26]35	[26]35	[27]48	NP	1	NP	NP	18,000	[28]18,900	32,000	[28]33,600	Spec
18	Louisiana	96	[6]12-6	35	[19]40	50	60	1	1	NP	18,000		32,000		Axle
19	Maine	96	[30]12-6	50	50	50	50	1	1	NP	22,000		32,000		Tabl
20	Maryland	96	[6]12-6	55	55	55	55	NR	NR	NR	22,400		[31]40,000		Form
21	Massachusetts	96	NR	35	[19]40	45	NP	1	NP	NP	22,400		36,000		Tabl
22	Michigan	96	[6]12-6	35	40	55	55	1	1	2	[33]18,000		[34]32,000		Axle
23	Minnesota	96	[6]12-6	40	40	50	50	1	1	NP	18,000		32,000		Tabl
24	Mississippi	96	[6]12-6	35	40	[36]45	45	1	1	NP	18,000		28,650	[35]32,000	Tabl
25	Missouri	96	12-6	35	40	45	45	1	1	2	18,000		32,000		Tabl
26	Montana	[18]96	13-6	35	40	60	60	1	1	[37]2	18,000		32,000		Tabl
27	Nebraska	96	13-6	35	[19]40	50	50	1	1	NP	18,000	18,900	32,000	33,600	Tabl
28	Nevada	96	NR	NR	NR	NR	NR	NR	NR	NR	18,000	18,900	32,000	33,600	Tabl
29	New Hampshire	96	13-6	35	[19]40	45	45	NR	NR	NR	22,400		36,000		Tabl
30	New Jersey	96	13-6	35	[39]35	45	[40]50	1	1	NP	22,400	23,520	32,000	33,600	Spec
31	New Mexico	[41]96	13-6	40	40	65	65	1	1	2	21,600		34,320		Tabl
32	New York	96	13-0	35	[42]35	50	50	1	1	NP	22,400		36,000		Form
33	North Carolina	96	[6]12-6	35	[19]40	[43]48	[43]50	1	1	NP	18,000	19,000	36,000	38,000	Spec
34	North Dakota	96	13-6	[14]35	[19]40	50	50	1	1	NP	18,000		30,000		Form
35	Ohio	96	[6]12-6	35	[19]40	50	60	1	NR	NR	19,000		[44]31,500		Form
36	Oklahoma	96	13-6	35	45	50	50	1	1	NP	18,000		32,000		Tabl
37	Oregon	96	[45]12-6	35	[35]40	[46,35]55	[35]60	1	1	[35]2	[47]18,000		[47]32,000		Tabl
38	Pennsylvania	96	[6]12-6	35	[19]40	50	50	1	1	NP	22,400	23,072	36,000	37,080	Spec
39	Puerto Rico	98	12-0	NS	NS	NS		NS	NS	NS	NS		NS		Spec
40	Rhode Island	102	12-6	40	40	50	50	1	1	NP	22,400		NS		Spec
41	South Carolina	96	[54]12-6	[14]35	[19]40	50	60	1	1	NP	20,000		32,000		Tabl
42	South Dakota	96	13-0	35	40	50	60	1	1	NP	18,000		32,000		Tabl
43	Tennessee	96	12-6	35	40	45	45	1	[24]1	NP	18,000		32,000		Tabl
44	Texas	96	13-6	35	40	50	50	1	1	NP	18,000	18,900	32,000	33,600	Tabl
45	Utah	96	14-0	45	45	60	60	NR	NR	NR	18,000		33,000		Tabl
46	Vermont	96	12-6	50	50	50	50	1	1	NP	NS		NS		Spec
47	Virginia	96	[6]12-6	35	[35]40	50	50	1	1	NP	18,000		[56]32,000		Tabl
48	Washington	96	[5]12-6	35	[19]40	60	60	1	1	[57]2	18,000	[58]18,500	32,000	[58]33,000	Tabl
49	West Virginia	96	[6]12-6	35	[19]40	50	50	1	1	NP	18,000	18,900	32,000	33,600	Tabl
50	Wisconsin	96	[6]12-6	35	40	50	50	1	1	NP	18,000	[60]19,500	30,000	32,000	Tabl
51	Wyoming	96	13-6	40	40	60	60	1	1	2	18,000		32,000	[62]36,000	Tabl
	AASHO Policy	96	12-6	35	[19]40	50	60	1	1	NP	18,000		32,000		Tabl
Number of States	Higher	4	23	16	29	15	6	6	8	21	31		30		For
	Same	47	27	35	18	26	12	45	38	30	20		18		Tab
	Lower	0	1	0	4	10	33	0	5	0	0		3		Spe

NP—Not permitted. NR—Not restricted. NS—Not specified.

[1] Various exceptions for farm and construction equipment; public utility vehicles; urban, suburban, and school buses; haulage of agricultural and forest products; at wheels of vehicles; for safety accessories, and on designated highways.

[2] Various exceptions for utility vehicles and loads.

[3] When not specified, limited to number possible in practical combinations within permitted length limits.

[4] Legally specified or established by administrative regulation.

[5] Computed under the following conditions to permit comparison on a uniform basis between States with different types of regulation:

A. Front axle load of 8,000 pounds.

B. Maximum practical wheelbase within applicable length limits:

(1) Minimum front overhand of 3 feet.

(2) In the case of a 4-axle truck-tractor semitrailer, rear overhand computed as necessary to distribute the maximum possible uniform load on the maximum permitted length of semitrailer to the single drive-axle of the tractor and to the tandem axles of the semitrailer, within the permitted load limits of each.

(3) In the case of a combination having 5 or more axles, minimum possible combined front and rear overhand assumed to be 5 feet, with maximum practical load on maximum permitted length of semitrailer, subject to control of loading on axle groups and on total wheelbase as applicable.

C. Including statutory enforcement tolerances as applicable.

[6] Auto transports 13 feet 6 inches.

[7] Does not apply to combinations of adjacent load-carrying single axles.

[8] 56,000 pounds on load-carrying axles, exclusive of steering-axle load.

[9] On specific routes in urban or suburban service under special permit from P.U.C. 40 feet, also 3-axle buses with turning radius less than 45 feet without restriction.

[10] Limited by 40-foot maximum length of semitrailer to 55-foot practical maximum length in combination.

[11] Buses 102 inches.

[12] Truck tractor and housetrailer 50 feet.

[13] Legal limit 60,000 pounds.

[14] Three-axle vehicles 40 feet.

[15] Truck 39 feet 6 1/2 inches; bus 45 feet 2

[16] 63,280 pounds maximum, except on roads un

[17] 700 (L+40) when L is 18' or less; 800 (L+ structures with span of 20' or over.

[18] Buses 102 inches on highways of surfaced

[19] Less than three axles 35 feet.

[20] Special limits for vehicles hauling timbe cultural products including livestock; single vehicle with 3 or 4 axles permitted 66,000 pou permitted 79,000 pounds maximum at 43-foot axl

[21] On designated highways; 16,000 pounds on

[22] Without tandem axles 45,000 pounds.

[23] On designated highways; single axle 22,40 total of all excesses of weight under one or m

[24] Limited to 3,500 pounds.

[25] On designated highways; 11 feet 6 inches

[26] On designated highways; trucks 26.5 feet

[27] Class AA highways; 45 feet on other highwa

[28] Class AA highways only.

[29] Maximum gross weight on Class A highways

[30] Including load 14 feet.

[31] Spaced less than 48 inches 36,000 pounds.

[32] Subject to axle and tabular limits.

[33] Single axle spaced less than 9 feet from

[34] On designated highways only and limited t

[35] On designated highways only.

[36] Auto transports permitted 50 feet.

[37] Semitrailer and semitrailer converted to f

[38] Dual-drive axles; otherwise 40,000 pounds.

D WEIGHTS COMPARED WITH AASHO STANDARDS

oads, July 1, 1957

s weight limit			Specified maximum gross weight—pounds[4]						Practical maximum gross weight—pounds[5]						Line
	Applicable to:		Truck		Truck tractor semitrailer				Truck		Truck-tractor semitrailer				
Formula or equation	Any group of axles	Total wheel base only	2-axle	3-axle	3-axle	4-axle	5-axle	Other combination	2-axle	3-axle	3-axle	4-axle	5-axle	Other combination	
		X							27,800	47,600	47,600	60,010	64,650	NP	1
	Under 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,800	2
									26,500	40,500	45,000	59,000	65,000	65,000	3
	Under 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,000	4
800 (L + 40)		X	30,000	46,000					26,000	44,000	44,000	62,000	75,000	76,000	5
			32,000	50,000	50,000	60,000	60,000	NP	30,848	44,720	51,000	61,200	61,200	NP	6
	X		30,000	40,000	48,000	60,000	60,000	60,000	28,000	40,000	48,000	56,350	60,000	60,000	7
	X								30,000	46,000	52,000	58,450	61,490	64,650	8
		X							30,000	52,000	52,000	65,200	71,115	71,115	9
								63,280	28,340	48,680	48,680	63,280	63,280	63,280	10
0 & 800 (L + 40)	Under 18'	Over 18'							32,000	38,800	56,000	62,800	69,600	78,000	11
	X								26,000	40,000	44,000	58,000	72,000	76,800	12
			36,000	[22]41,000	45,000	59,000	68,000	72,000	26,000	40,000	44,000	58,000	68,000	72,000	13
								72,000	27,000	41,000	45,000	59,000	73,000	73,000	14
	X								26,540	40,960	45,080	59,500	68,657	NP	15
	X								26,000	40,000	44,000	55,470	63,890	63,890	16
			36,000	50,000	54,000	59,640	59,640	NP	26,900	41,600	45,800	59,640	59,640	NP	17
									26,000	40,000	44,000	58,000	72,000	76,000	18
	X		32,000	50,000	50,000	60,000	60,000	60,000	30,000	40,000	52,000	60,000	60,000	60,000	19
850 (L + 40)		X					65,000	65,000	30,400	48,000	52,800	65,000	65,000	65,000	20
1,000 (L + 25)		X	[32]46,000	[32]60,000	[32]60,000	[32]60,000	[32]60,000	NP	30,400	44,000	52,800	60,000	60,000	NP	21
									26,000	[30]40,000	44,000	[35]58,000	[35]66,000	[35]102,000	22
	X								26,000	40,000	44,000	58,000	68,000	72,500	23
	X								26,000	36,650	44,000	[35]54,650	[35]55,980	[35]55,980	24
	X								26,000	40,000	44,000	55,470	64,650	64,650	25
	Under 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,000	26
	Under 22'	Over 22'							26,780	41,200	45,320	57,134	66,590	66,590	27
	Under 18'	Over 18'							26,900	41,600	45,800	60,000	74,000	76,800	28
		X	33,400	[38]47,500	52,800	66,400			30,400	44,000	52,800	66,400	66,400	66,400	29
			30,000	40,000	60,000	60,000	60,000	60,000	31,500	41,600	55,040	63,000	63,000	63,000	30
	Under 18'	Over 18'							29,600	42,320	51,200	63,920	76,640	86,400	31
34,000 + 850L		X					65,000	65,000	30,400	44,000	52,800	65,000	65,000	65,000	32
			31,500	46,200	46,200	58,800	58,800	58,800	27,000	46,000	46,000	58,800	58,800	58,800	33
& 750 (L + 40)	Under 18'	Over 18'							26,000	38,000	44,000	56,000	63,750	63,750	34
800 (L + 47.5)		X							27,000	39,500	46,000	58,500	71,000	78,000	35
	X								26,000	40,000	44,000	55,470	60,000	60,000	36
	Under 18'	Over 18'					72,000	76,000	26,000	40,000	44,000	58,000	72,000	76,000	37
			33,000	47,000	50,000	60,000	60,000	62,000	31,072	45,080	51,500	61,800	61,800	63,860	38
															39
			[50]36,000	[51]44,000	[52]50,000	[53]60,000	60,000	88,000	30,400	44,000	50,000	60,000	60,000	88,000	40
	X								28,000	40,000	48,000	60,000	66,839	71,115	41
	X								26,000	40,000	44,000	58,000	72,000	73,280	42
		X							26,000	40,000	44,000	55,980	55,980	43,500	43
	X								26,900	41,600	45,800	57,844	61,340	61,340	44
	X								26,000	41,000	44,000	59,000	74,000	79,900	45
			30,000	40,000	50,000	[55]60,000	[55]60,000	[55]60,000	30,000	40,000	50,000	60,000	60,000	60,000	46
	X						[35]56,800	[35]56,800	26,000	40,000	44,000	56,800	56,800	56,800	47
	Under 18'	Over 18'	28,000	36,000	46,000	60,000	68,000	72,000	26,000	36,000	44,000	60,000	68,000	72,000	48
	X								26,900	41,600	45,800	57,844	63,840	63,840	49
1,000 (L + 26)	X								27,500	40,000	47,000	59,500	68,000	68,000	50
	X								26,000	44,000	44,000	62,000	73,000	73,000	51
$(L + 24) - 3L^2$	X								26,000	40,000	44,000	55,470	61,490	71,900	
	18	19							29 21 0	25 20 5	29 21 0	45 4 1	35 2 13	20 0 30	

56,000 pounds maximum.
18': 900 (L+ 40) on highways having no

otherwise as administratively authorized.

s, concentrates, aggregates, and agri-
m axle 37,800 pounds, gross weight table:
le spacing, vehicle with 5 or more axles

00 pounds; tolerance of 1,000 pounds on
oad and gross weight.

highways.

highways 30,000 pounds.

,000 pounds.
nation; otherwise 26,000 pounds.

dolly.

[39] Or as prescribed by P.U.C.

[40] Exception for poles, pilings, structural units, ec., permitted 70 feet.

[41] On designated highways 102 inches.

[42] Trackless trolleys and buses 7 passengers or more, P.S.C. certificate 40 feet.

[43] Including front and rear bumpers.

[44] Spaced less than 4 feet 24,000 pounds.

[45] Certain types of vehicles and commodities under special permit on designated highways up to 13 feet 6 inches.

[46] 60 feet allowed truck tractor semitrailer on 4 major Interstate routes.

[47] Logging vehicles permitted 3-foot wheelbase tolerance, 19,000-pound single axle, 34,000-pound tandem axle.

[48] Governs gross weight permitted on highways designated by resolution of State highway commission.

[49] Single unit truck with 4 axles permitted 60,000 pounds.

[50] Axles spaced less than 6 feet 32,000 pounds; less than 12 feet 36,000 pounds; 12 feet or more gross weight governed by axle limit.

[51] Single vehicle with 3 or more axles spaced less than 16 feet 40,000 pounds; less than 20 feet 44,000 pounds; 20 feet or more governed by axle limit.

[52] Tractor semitrailer with 3 or more axles spaced less than 22 feet 46,000 pounds; not less than 27 feet 50,000 pounds.

[53] Axles spaced 27 feet or more.

[54] Auto transports and vehicles transporting materials for Department of Defense on highways with adequate clearance 13' -6".

[55] Tandem axles on trailer equipped with adequate brakes.

[56] Vehicles registered before July 1, 1956, permitted limits in effect January 1, 1956, for life of vehicle.

[57] Under State highway commission rules.

[58] Within discretion of enforcement officer.

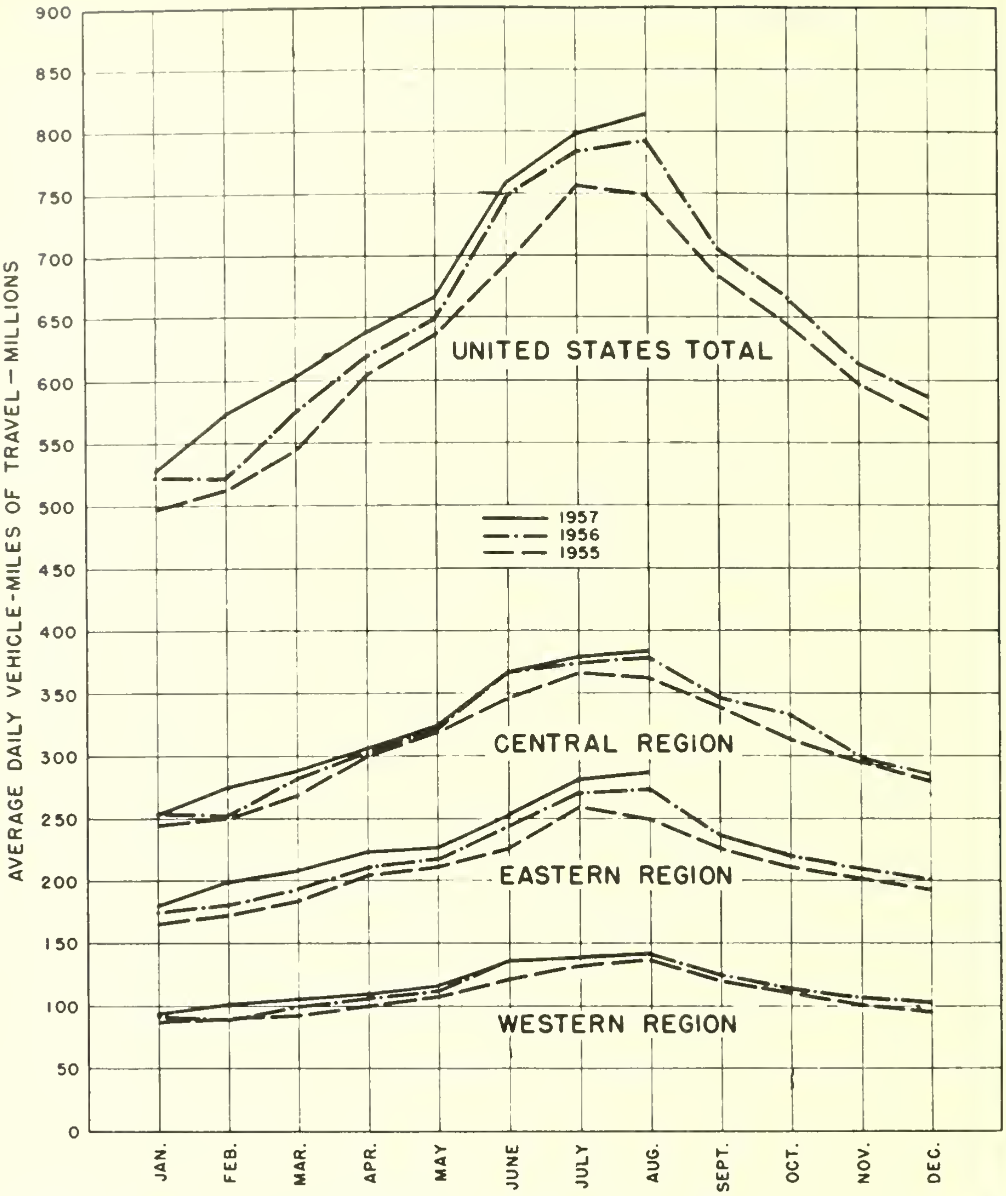
[59] Vehicles hauling logs permitted wheelbase and gross weight tolerances. Discretionary enforcement tolerances not included in computation of practical maximum gross weights.

[60] Axle load 21,000 pounds on 2-axle trucks hauling unmanufactured forest products.

[61] On Class A highways.

[62] Based on ruling of Attorney General.

Figure 3 (Right).—Average daily travel on main rural roads in 1955, 1956, and in the first 8 months of 1957.



(Continued from page 255)

is as follows: eastern region, 220 and 209 million vehicle-miles; central region, 316 and 308 million vehicle-miles; and western region, 114 and 108 million vehicle-miles, respectively. Percentage increases in 1956 over 1955 for the three regions were 5.3 percent for the eastern, 2.6 percent for the central, and 5.6 percent for the western.

Table 3 provides a comparison of the estimated vehicle-miles of travel on main rural roads by 5-year periods from 1936 through 1956. From the data given in this table it is seen that the vehicle-mileages traveled by major classes of vehicles were approximately the same in 1946 as in 1941. However, during the war period single-unit truck travel decreased 6 percent, whereas travel by truck combinations increased 27 percent.

In the 5 years that followed 1946, traffic by all classes of vehicles increased faster than in any other similar period. The annual average growth rates were as follows: all vehicles, 10.4 percent; passenger cars and buses, 9.8 percent; all trucks and truck combinations, 13.2 percent; single-unit trucks, 11.2 percent; and truck combinations, 19 percent.

In the last 5-year period (1951–56), the average annual increases, although showing a steady gain, were generally much smaller than those of the previous 5-year periods. The annual growth rates were as follows: all vehicles, 5.0 percent; passenger cars and buses, 5.2 percent; trucks and truck combinations, 4.8 percent; single-unit trucks, 4.4 percent; and truck combinations, 5.4 percent.

Over the 20-year period, 1936–56, travel by all vehicles increased an average amount each year of 8.4 percent; passenger cars and buses, 7.8 percent; all trucks and combinations, 11.2 percent; single-unit trucks, 8.4 percent; and truck combinations, 24.4 percent.

A comparison of 1956 travel on main rural roads with 1955 data previously published reveals the following increases in 1956: all vehicles, 4.3 percent; passenger cars, 4.0 percent; all trucks and truck combinations, 5.5 percent; single-unit trucks, 5.2 percent; and truck combinations, 6.0 percent.

An interesting and important observation, as shown in table 3, is that the percentage of travel by combination vehicles, in relation to total truck travel, increased more rapidly in the 10-year period, 1936–46, than in the period, 1946–56. In 1936, about 18 percent of the travel of all freight-carrying vehicles was by combination types, while in 1946, the percentage had increased to approximately 27 percent. In 1951 and 1956, the percentages were 31 and 32, respectively.

Distribution of Traffic Within Census Divisions

The percentage distribution of travel, by vehicle types, on main rural roads within each census division is given in table 4. The table shows that the largest percentage of passenger-car travel in relation to total travel within a division was in the Pacific division, with the New England division following closely. A comparison of truck travel shows that the East South Central division has the highest percentage, with the West South Central and Mountain divisions following in order. The lowest percentages for truck travel were found in the Pacific and New

Table 3.—Comparison of estimated vehicle-miles of travel on main rural roads in 1936, 1941, 1946, 1951, and 1956

Year	Total travel, vehicle-miles	Passenger-car and bus travel		Truck and truck-combination travel		Single-unit truck travel		Truck-combination travel	
		Vehicle-miles	Percentage of total travel	Vehicle-miles	Percentage of total travel	Vehicle-miles	Percentage of total truck travel	Vehicle-miles	Percentage of total truck travel
	Millions	*Millions*		*Millions*		*Millions*		*Millions*	
1936	88,412	73,005	82.57	15,407	17.43	12,650	82.11	2,757	17.89
1941	122,721	98,509	80.27	24,212	19.73	19,067	78.75	5,145	21.25
1941:1936 ratio	*1.39*	*1.35*	*.97*	*1.57*	*1.13*	*1.51*	*1.96*	*1.87*	*1.19*
1946	124,373	99,985	80.39	24,388	19.61	17,850	73.19	6,538	26.81
1946:1941 ratio	*1.01*	*1.01*	*1.00*	*1.01*	*.99*	*.94*	*.93*	*1.27*	*1.26*
1946:1936 ratio	*1.41*	*1.37*	*.97*	*1.58*	*1.13*	*1.41*	*.89*	*2.37*	*1.50*
1951	189,650	149,109	78.62	40,541	21.38	27,810	68.60	12,731	31.40
1951:1946 ratio	*1.52*	*1.49*	*.98*	*1.66*	*1.09*	*1.56*	*.94*	*1.95*	*1.17*
1951:1941 ratio	*1.55*	*1.51*	*.98*	*1.67*	*1.08*	*1.46*	*.87*	*2.47*	*1.48*
1951:1936 ratio	*2.15*	*2.04*	*.95*	*2.63*	*1.23*	*2.20*	*.84*	*4.62*	*1.76*
1956	237,748	187,678	78.94	50,070	21.06	33,842	67.59	16,228	32.41
1956:1951 ratio	*1.25*	*1.26*	*1.00*	*1.24*	*.99*	*1.22*	*.99*	*1.27*	*1.03*
1956:1946 ratio	*1.91*	*1.88*	*.98*	*2.05*	*1.07*	*1.90*	*.92*	*2.48*	*1.21*
1956:1941 ratio	*1.94*	*1.91*	*.98*	*2.07*	*1.07*	*1.77*	*.86*	*3.15*	*1.53*
1956:1936 ratio	*2.69*	*2.57*	*.96*	*3.25*	*1.21*	*2.68*	*.82*	*5.89*	*1.81*

Table 4.—Percentage distribution of travel, by vehicle types for each census division, on main rural roads in the summer of 1956 with comparative data by vehicle types for 1951

Vehicle type	Eastern region				Central region					Western region			United States average, 1956	United States average, 1951	1956:1951 ratio of vehicle-miles traveled
	New England division	Middle Atlantic division	South Atlantic division	Average	East North Central division	East South Central division	West North Central division	West South Central division	Average	Mountain division	Pacific division	Average			
Passenger cars:															
Local	60.46	59.71	58.73	59.33	57.62	52.14	62.32	58.99	58.01	75.24	42.24	62.99	59.33	59.26	1.25
Foreign	22.23	17.28	20.37	19.54	22.09	19.38	17.67	14.59	18.87	32.91	8.41	17.50	18.86	18.43	1.28
All passenger cars	82.69	76.99	79.10	78.87	79.71	71.52	79.99	73.58	76.88	75.15	83.65	80.49	78.19	77.69	1.26
Single-unit trucks:															
Panel and pickup	3.57	8.72	7.41	7.31	5.21	9.68	6.00	11.35	7.64	10.16	7.30	8.36	7.66	7.00	1.37
Other 2-axle, 4-tire	2.64	1.37	.80	1.27	.19	.02	.35	.05	.16	1.06	.82	.91	.66	.62	1.35
Other 2-axle, 6-tire	5.25	5.19	5.32	5.27	5.49	8.07	5.67	5.61	5.98	6.53	1.83	3.58	5.32	6.62	1.01
3-axle	.60	.63	.72	.67	.57	.51	.50	.18	.46	.98	.73	.82	.59	.43	1.72
All single-unit trucks	12.06	15.91	14.25	14.52	11.46	18.28	12.52	17.19	14.24	18.73	10.68	13.67	14.23	14.67	1.22
Truck-tractor and semitrailer combinations:															
3-axle	3.59	4.30	1.84	2.97	2.50	3.53	1.94	2.77	2.62	1.49	.62	.94	2.44	3.52	.87
4-axle	.97	1.77	3.77	2.65	4.33	5.59	3.62	5.17	4.60	1.26	.61	.85	3.29	2.22	1.86
5-axle or more	.01	.02	.02	.02	1.00	.03	1.18	.50	.75	1.93	1.12	1.42	.62	.50	1.54
All truck-tractor and semitrailer combinations	4.57	6.09	5.63	5.64	7.83	9.15	6.74	8.44	7.97	4.68	2.35	3.21	6.35	6.24	1.27
Combinations involving full trailers:															
4-axle or less	.01	.27	.01	.10	.06	----------	.18	.05	.07	.20	.12	.15	.10	.14	.85
5-axle	----------	.01	.01	.01	.31	----------	.08	.01	.14	.53	1.84	1.36	.31	.21	1.87
6-axle or more	----------	----------	----------	------	.10	----------	(1)	----------	.04	.04	.46	.30	.07	.12	.75
All combinations involving full trailers	.01	.28	.02	.11	.47	----------	.26	.06	.25	.77	2.42	1.81	.48	.47	1.27
All combinations	4.58	6.37	5.65	5.75	8.30	9.15	7.00	8.50	8.22	5.45	4.77	5.02	6.83	6.71	1.26
All trucks and combinations	16.64	22.28	19.90	20.27	19.76	27.43	19.52	25.69	22.46	24.18	15.45	18.69	21.06	21.38	1.24
Buses	.67	.73	1.00	.86	.53	1.05	.49	.73	.66	.67	.90	.82	.75	.93	1.01
All vehicles	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	100.00	1.25

[1] Less than 0.005 percent.

England divisions. This condition naturally follows the higher percentage of passenger-car travel in these two areas. On a percentage basis, travel by all types of truck combinations in the East South Central division exceeded all other divisions and was followed closely by the West South Central division. When the comparison is restricted to truck and full-trailer combinations, it can readily be seen that travel by this vehicle type is heavily concentrated in the Pacific division particularly, and also in the Mountain and East North Central divisions.

When the percentage distribution of travel by types of vehicles on main rural roads in 1956, as given in table 4, is compared with a similar distribution for 1951, it is found that during the 5-year period, passenger-car travel and truck-combination travel related to total travel increased slightly; while that of single-unit trucks decreased.

The ratios of 1956 traffic on main rural roads to corresponding traffic in 1951 for each type of vehicle is given in the right-hand column of table 4. For the United States as a whole, travel increased 25 percent in 1956 compared with 1951, an average annual increase of 5.0 percent. During the period, passenger-car travel increased at the rate of 5.2 percent a year; single-unit trucks, 4.4 percent; truck combinations, 5.2 percent; and buses, 0.2 percent.

Weight Stations Operated

During the summer of 1956 a total of 506 loadometer or pitscale stations were operated in 43 States for the purpose of collecting data concerning vehicle types, weights, and loading

Table 5.—Comparison of vehicle-miles of travel, percentage distribution of travel, percentage of vehicles loaded, average weight of loaded vehicles, percentage of vehicles empty, average weight of empty vehicles, and average weight of loaded and empty vehicles on main rural roads, by vehicle types, in 1956 and 1951

Vehicle type	Vehicle-miles of travel		Percentage of vehicle-miles traveled		Percentage loaded		Average weight of loaded vehicles		Percentage empty		Average weight of empty vehicles		Average weight of loaded and empty vehicles	
	1956	1951	1956	1951	1956	1951	1956	1951	1956	1951	1956	1951	1956	1951
Single-unit trucks:	*Millions*	*Millions*	*Percent*	*Percent*	*Percent*	*Percent*	*Pounds*	*Pounds*	*Percent*	*Percent*	*Pounds*	*Pounds*	*Pounds*	*Pounds*
Panel and pickup	18,208	13,281	36.37	32.76	42.83	38.95	5,307	5,498	57.17	61.05	4,196	4,210	4,672	4,712
Other 2-axle, 4-tire	1,582	1,176	3.16	2.90	59.10	54.59	6,565	6,522	40.90	45.41	5,078	4,924	5,958	5,796
Other 2-axle, 6-tire	12,653	12,541	25.27	30.94	60.74	57.99	14,466	14,060	39.26	42.01	8,589	8,137	12,159	11,572
3-axle	1,399	812	2.79	2.00	59.11	60.84	30,790	29,924	40.89	39.16	15,553	15,187	24,557	24,153
Total or average	33,842	27,810	67.59	68.60	50.96	48.84	10,678	11,024	49.04	51.16	5,936	5,937	8,353	8,421
Truck-tractor and semitrailer combinations:														
3-axle	5,816	6,684	11.61	16.49	65.91	66.70	32,561	33,894	34.09	33.30	18,477	17,918	27,760	28,574
4-axle	7,814	4,208	15.61	10.38	70.41	71.65	47,640	47,266	29.59	28.35	24,109	23,337	40,677	40,482
5-axle or more	1,472	955	2.94	2.35	77.99	75.71	60,517	60,455	22.01	24.29	29,396	29,504	53,664	52,937
Total or average	15,102	11,847	30.16	29.22	69.41	69.18	43,535	41,373	30.59	30.82	22,062	20,282	36,966	34,873
Truck and trailer combinations:														
4-axle or less	229	269	.46	.66	76.86	68.40	46,844	42,545	23.14	31.60	18,933	26,293	40,424	37,409
5-axle	727	389	1.45	.96	71.66	59.64	65,742	65,594	28.34	40.36	27,954	28,303	55,029	50,543
6-axle or more	170	226	.34	.56	68.24	70.35	71,924	65,899	31.76	29.65	33,722	31,039	59,718	55,563
Total or average	1,126	884	2.25	2.18	72.20	65.04	62,520	58,599	27.80	34.96	27.437	28,325	52,767	48,015
Total or average, all combinations	16,228	12,731	32.41	31.40	69.60	68.89	44,900	42,501	30.40	31.11	22.403	20,911	38,063	35,784
Total or average, all trucks and combinations	50,070	40,541	100.00	100.00	57.00	55.13	24,221	23,376	43.00	44.87	9,708	9,197	17,980	17,014

Table 6.—Comparison of estimated percentage of trucks loaded, average carried load, and ton-miles carried on main rural roads in 1936, 1941, 1946, 1951, and 1956

Year	All trucks and truck combinations			Single-unit trucks			Truck combinations		
	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried	Percentage loaded	Average weight of carried load	Ton-miles carried
		Tons	*Millions*		*Tons*	*Millions*		*Tons*	*Millions*
1936	62.8	2.90	28,005	60.7	1.86	14,258	72.2	6.90	13,747
1941	66.7	3.64	58,853	65.4	2.29	28,505	71.6	8.24	30,348
1941:1936 ratio	*1.06*	*1.26*	*2.10*	*1.08*	*1.23*	*2.00*	*.99*	*1.19*	*2.21*
1946	51.7	4.84	61,084	46.4	2.31	19,117	66.2	9.70	41,967
1946:1941 ratio	*.78*	*1.33*	*1.04*	*.71*	*1.01*	*.67*	*.92*	*1.18*	*1.38*
1946:1936 ratio	*.82*	*1.67*	*2.18*	*.76*	*1.24*	*1.34*	*.92*	*1.41*	*3.05*
1951	55.1	5.66	126,402	48.8	2.31	31,396	68.9	10.83	95,006
1951:1946 ratio	*1.07*	*1.17*	*2.07*	*1.05*	*1.00*	*1.64*	*1.04*	*1.12*	*2.26*
1951:1941 ratio	*.83*	*1.55*	*2.15*	*.75*	*1.01*	*1.10*	*.96*	*1.31*	*3.13*
1951:1936 ratio	*.88*	*1.95*	*4.51*	*.80*	*1.24*	*2.20*	*.95*	*1.57*	*6.91*
1956	57.0	6.00	171,249	51.0	2.39	41,248	69.6	11.51	130,001
1956:1951 ratio	*1.03*	*1.06*	*1.35*	*1.05*	*1.03*	*1.31*	*1.01*	*1.06*	*1.37*
1956:1946 ratio	*1.10*	*1.24*	*2.80*	*1.10*	*1.03*	*2.16*	*1.05*	*1.19*	*3.10*
1956:1941 ratio	*.85*	*1.65*	*2.91*	*.78*	*1.04*	*1.45*	*.97*	*1.40*	*4.28*
1956:1936 ratio	*.91*	*2.07*	*6.11*	*.84*	*1.28*	*2.89*	*.96*	*1.67*	*9.46*

practices. During this survey all vehicles (passenger cars, single-unit trucks, truck combinations, and buses) were counted at these stations and freight-carrying vehicles were classified according to number of axles and tire equipment. Approximately 428,000 trucks and truck combinations passed the stations of which about 138,000 were weighed. A record was made of the type of vehicle, the weight of its axles, the spacing between axles, and whether the vehicle was loaded or empty

The stations used in the 1956 survey were located at the same points as in former years, most of them being at sites operated in the original surveys in the 1936–40 period. From comparable data collected at these locations. trends in travel, loading practices, and carried loads were obtained, which when applied to earlier estimates derived from comprehensive surveys, gave current estimates of annual vehicle-miles traveled by loaded vehicles and the carried load. The product of these two factors is the ton-miles of carried load. Data concerning the frequency of overloading and of heavy axle and heavy gross-weight occurrence also were made available. The remaining tables and charts in this article have been calculated by means of these trends, or by combining the actual data gathered in the summer survey with vehicle-mileage data developed from trends.

Table 5 provides considerable information concerning the vehicle-miles of travel by types of freight-carrying vehicles on main rural roads in 1956 and comparative data for 1951; also included are the percentage of travel of each vehicle type to the total travel of all types of trucks and truck combinations, the percentage carrying loads, the percentage traveling empty, the average loaded weight, the average empty weight, and the average weight of loaded and empty vehicles.

From table 5, it is seen that 57 percent of all freight-carrying vehicles were loaded and weighed an average of 24,221 pounds; empty vehicles averaged 9,708 pounds; and the average of all vehicles, loaded and empty, was 17,980 pounds. Data for 1956, when compared with those for 1951, show that the weights of loaded single-unit trucks decreased 3.2 percent in the 5-year period; truck combinations increased 5.6 percent; and the weights of all loaded vehicles increased 3.6 percent. At the same time, the weights of all empty single-unit trucks remained approximately the same; empty truck combinations increased 7.1 percent; and the weights of all empty vehicles, trucks and truck combinations, increased 5.6 percent. The average empty weights of truck and trailer combinations with four or less axles show a substantial decline from 1951 to 1956. The increasing use of homemade farm trailers drawn by lightweight trucks and similar combinations accounts for most of the decrease.

The heaviest average weights of loaded single-unit trucks in 1956 were found in the New England division, with those in the South Atlantic division being slightly less. Weights of loaded truck combinations were found to be heaviest in the Pacific division with those in the Mountain division being slightly smaller. The heaviest average weights for loaded trucks and truck combinations of all types were recorded in the East North Central division, with the average in the Pacific and Mountain divisions slightly lower. At the other extreme, the lowest average

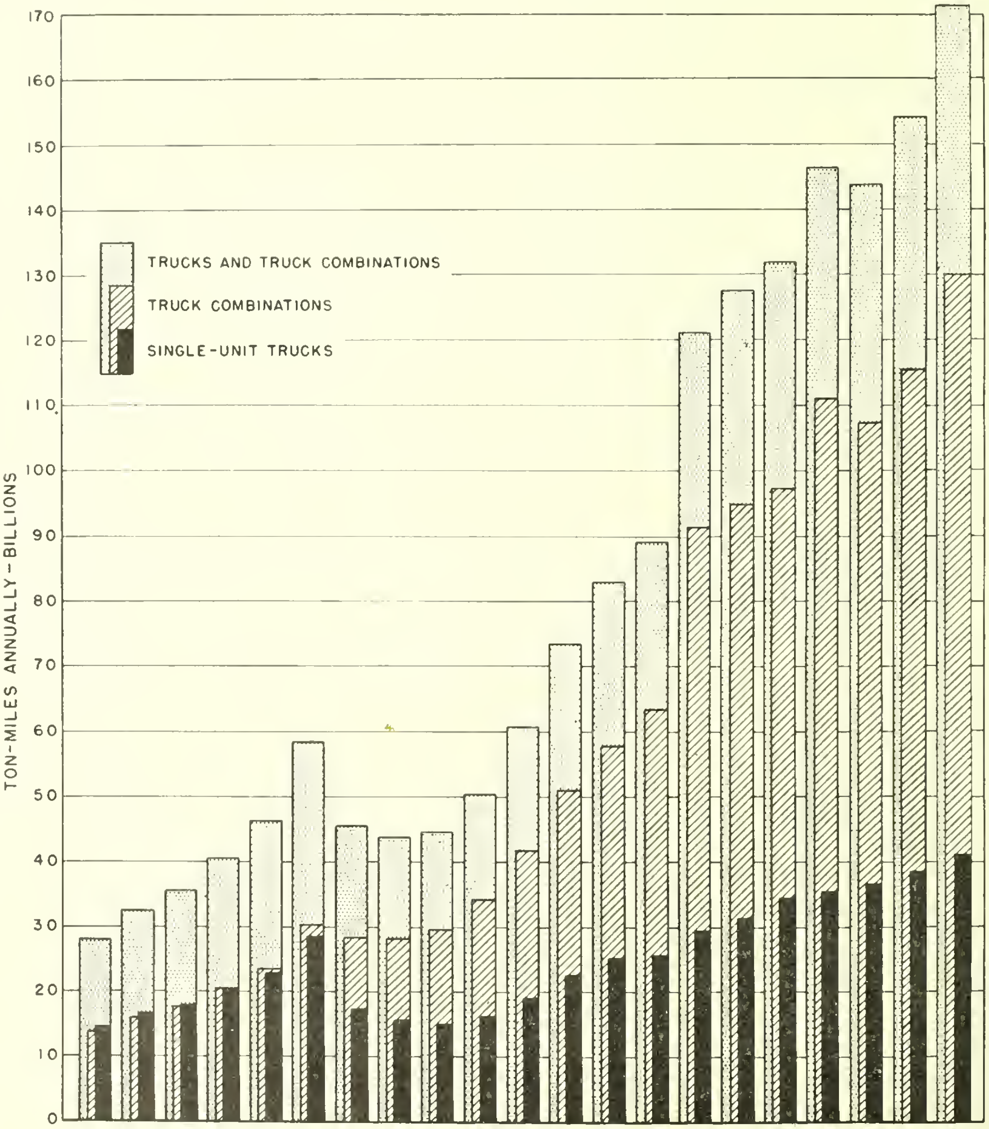


Figure 4.—Ton-miles carried by trucks and truck combinations on main rural roads, 1936–56.

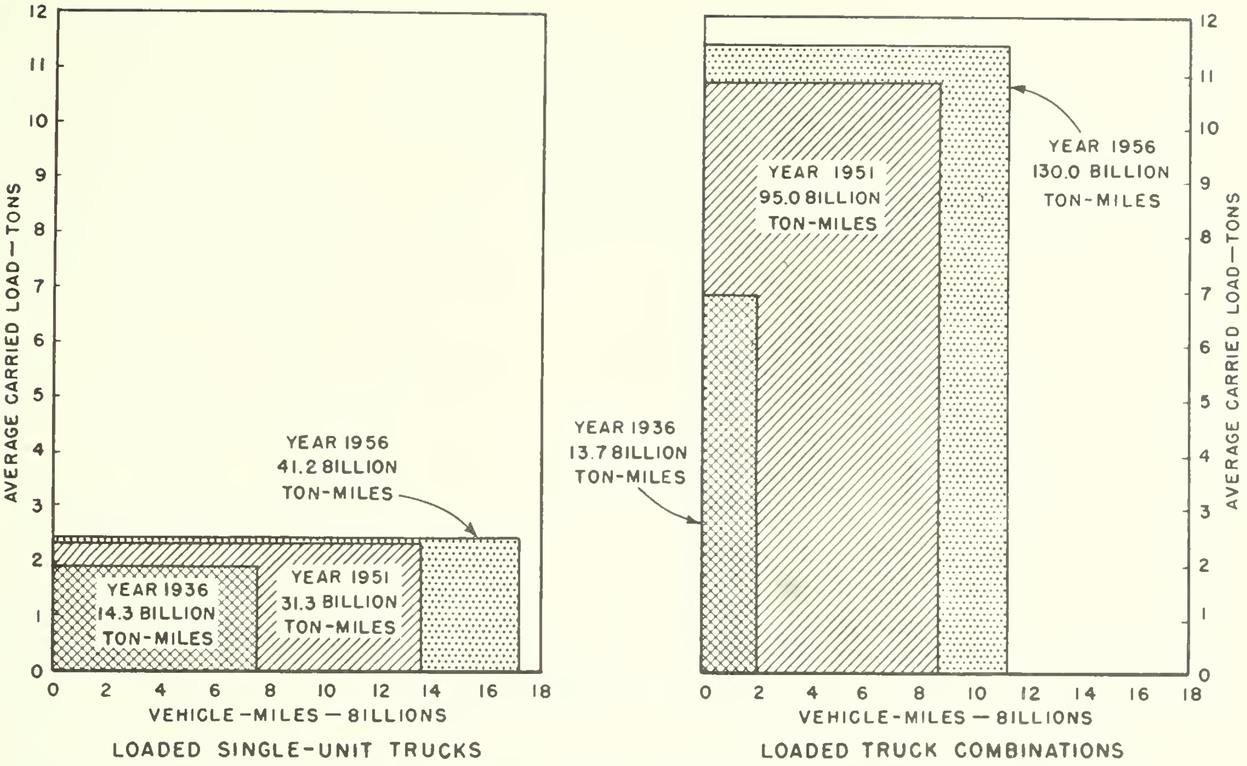


Figure 5.—Vehicle-miles of travel, average carried load, and ton-miles carried by trucks and truck combinations on main rural roads in 1956 compared with 1936 and 1951.

weights for all types of loaded vehicles were observed in the New England division. Average empty weights of vehicles followed an area distribution pattern similar to that for the average loaded weights.

Travel by Loaded Trucks Increases

The percentage of vehicles carrying loads has changed considerably since the beginning of the planning surveys, as indicated in table 6. From 1936 to 1941, between 60 and 65 percent of the single-unit trucks were loaded. When war was declared and driving restrictions were imposed, many small truck owners found it advantageous to use their vehicles for general transportation purposes. Thus it soon developed that the lighter weight trucks were being used more frequently for personal transportation than had previously been the case.

During World War II, the larger as well as the smaller single-unit trucks were used for the transportation of goods on less than half of their travel. Following the war, the popularity of the lightweight vehicles as a means of personal transportation continued, and in 1946 only about 46 percent of the travel of single-unit trucks involved carrying a load. Since 1946 the load factor has increased, and in 1956, for the first time since 1941, travel by loaded single-unit trucks exceeded that of empty vehicles of this type by a small margin.

For truck combinations, the relation of travel between loaded and empty vehicles has been fairly uniform throughout the 20-year period. In 1936 and 1941, 72 percent of the travel by these vehicles involved the carrying of goods or commodities as compared with 66 percent in 1946, 69 percent in 1951, and almost 70 percent in 1956. These comparisons may be noted more particularly in table 6 which gives, among other data, the percentages of loaded vehicles of the main freight-carrying types by 5-year periods commencing in 1936 and continuing through 1956.

Considering each type of truck and truck combination shown in table 5, it may be stated that the percentage that any type is

Table 7.—Comparison of percentage of travel by loaded vehicles, average carried load, relation of load to total loaded weight, ton-miles of carried load, and percentage of ton-miles carried, by vehicle types, in 1956 and 1951

Vehicle type	Percentage of travel by loaded vehicles		Average carried load		Relation of carried load to total loaded weight		Ton-miles of carried load		Percentage of ton-miles carried	
	1956	1951	1956	1951	1956	1951	1956	1951	1956	1951
Single-unit trucks:	*Percent*	*Percent*	*Tons*	*Tons*	*Percent*	*Percent*	*Millions*	*Millions*	*Percent*	*Percent*
Panel and pickup	27.32	23.14	0.75	0.70	28.3	25.5	5,882	3,611	3.44	2.86
Other 2-axle, 4-tire	3.28	2.87	1.08	.91	32.9	27.9	1,011	584	.59	.46
Other 2-axle, 6-tire	26.93	32.54	3.50	3.23	48.4	45.9	26,907	23,478	15.71	18.57
3-axle	2.90	2.21	9.01	7.54	58.5	50.4	7,448	3,723	4.35	2.95
Total or average	60.43	60.76	2.39	2.31	44.8	41.9	41,248	31,396	24.09	24.84
Truck-tractor and semitrailer combinations:										
3-axle	13.43	19.94	7.38	8.30	46.0	48.9	28,275	37,002	16.51	29.27
4-axle	19.27	13.49	12.32	12.42	52.6	52.5	67,799	37,460	39.59	29.64
5-axle or more	4.02	3.24	16.30	16.07	54.7	53.1	18,718	11,619	10.93	9.19
Total or average	36.72	36.67	10.95	10.50	51.1	50.7	114,792	86,081	67.03	68.10
Truck and trailer combinations:										
4-axle or less	.62	.82	14.63	8.45	55.0	40.5	2,575	1,554	1.50	1.23
5-axle	1.82	1.04	19.79	19.36	53.3	60.2	10,313	4,492	6.02	3.55
6-axle or more	.41	.71	20.01	18.10	49.1	56.0	2,321	2,879	1.36	2.28
Total or average	2.85	2.57	18.71	15.52	52.9	54.0	15,209	8,925	8.88	7.06
Total or average, all combinations	39.57	39.24	11.51	10.83	51.3	51.0	130,001	95,006	75.91	75.16
Total or average, all trucks and combinations	100.00	100.00	6.00	5.66	49.5	48.4	171,249	126,402	100.00	100.00

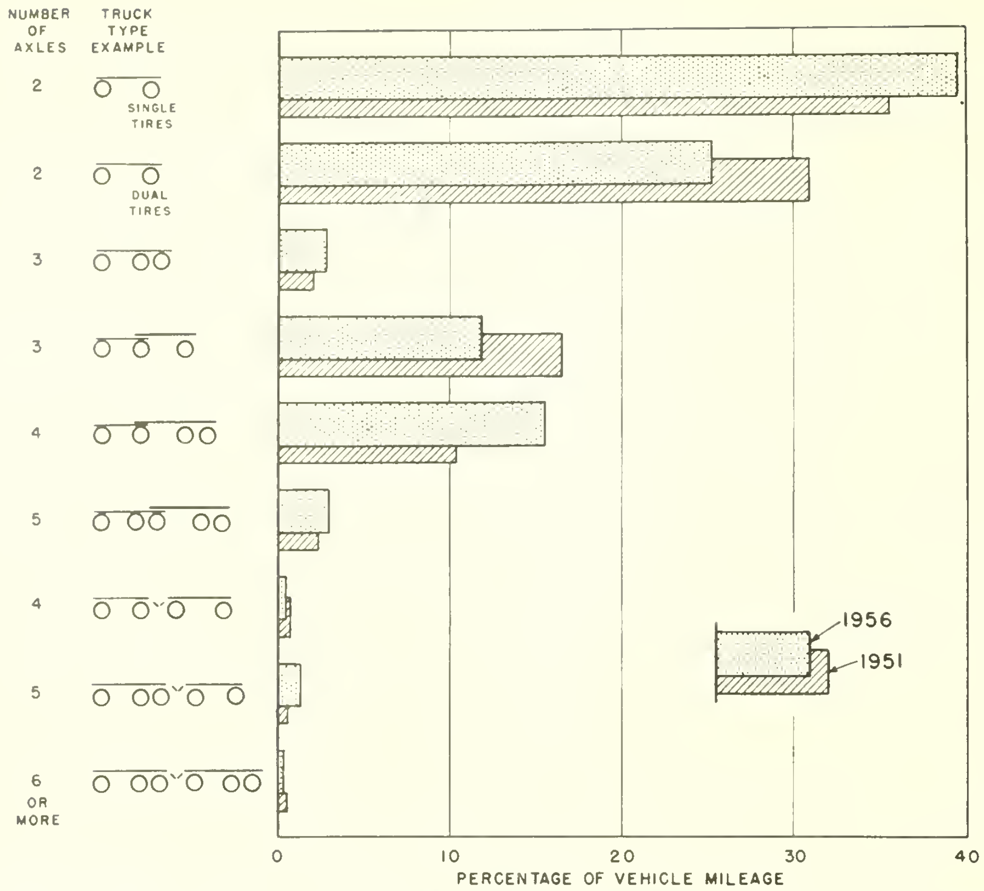


Figure 6.—Percentage of vehicle-miles traveled by various types of trucks and truck combinations on main rural roads in 1956 compared with 1951.

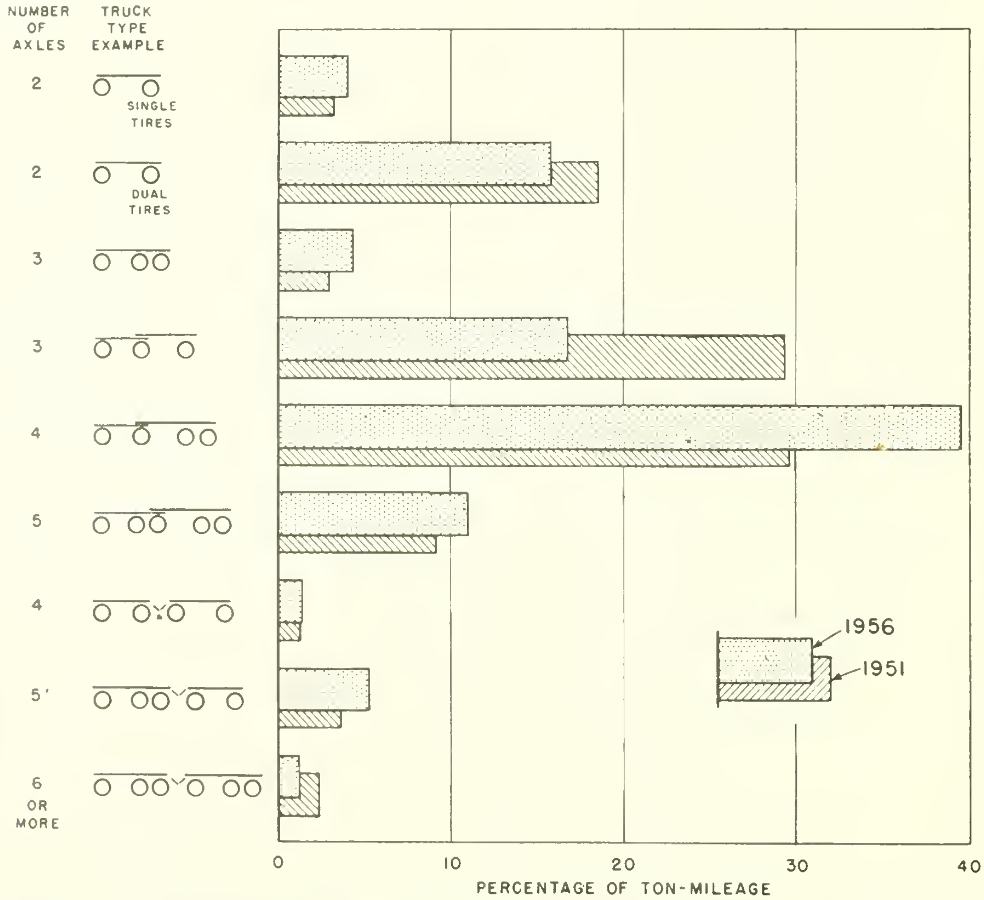


Figure 7.—Percentage of ton-miles hauled by various types of trucks and truck combinations on main rural roads in 1956 compared with 1951.

operated with a load increases roughly as the average weight increases. For instance, the 2-axle, 6-tire single-unit trucks, which averaged about 14,500 pounds loaded in 1956, were operated with a load about 61 percent of the time; the 3-axle tractor-semitrailer combinations, weighing over 32,500 pounds loaded, were operated with a load 66 percent of the time; and 4- and 5-axle combinations, weighing about 47,600 pounds and 60,500 pounds, were loaded 70 and 78 percent of the time, respectively.

Although the light panel and pickup trucks are used to a considerable extent for personal transportation, the slightly larger 2-axle trucks with single rear tires and carrying loads averaging about three-quarters of a ton were loaded 59 percent of the time, and surveys indicate little use for personal transportation. The same is true of the larger single-unit vehicles. Many of the truck combinations are common carriers which are less likely to be operated empty, inasmuch as they continually pick up and discharge freight. Combinations are found loaded in even greater proportion than the large single-unit trucks. Many of the latter type carry loads of a one-way variety, however, such as hauling building materials to construction projects, which tends to reduce the percentage loaded.

The variation in the percentage loaded of the several vehicle types, light and heavy, has an important effect on the distribution of vehicle-miles traveled by loaded vehicles and consequently on the ton-mileage of loads transported. A comparison of the percentages of vehicle-miles traveled by each vehicle type, as given in table 5, with the percentages of travel by each type of loaded vehicle, given in table 7, illustrates the wide differences between the total travel of these vehicles and the mileage of each when carrying a load.

Other interesting and useful information is given in table 7 concerning the average load in tons carried by each vehicle type and the relation in percentage points of the carried load to total loaded weight. From these data it may be observed that the average carried load and the relation of carried load to total load have generally increased in the 5-year period from 1951 through 1956. For instance, the tonnage carried by all types of tractor-semitrailer combinations increased from 10.50 tons in 1951 to 10.95 tons in 1956, while the relation of carried load to total weight increased from 50.7 percent in the earlier year to 51.1 percent in 1956. These increases, while small, indicate that larger payloads generally are being carried.

Volume of Freight Hauled

Ton-miles is the product of travel in vehicle-miles and the carried load in tons. The ton-mileages for each vehicle type traveling on main rural roads in 1956 and 1951 are given in table 7 with the corresponding percentage distribution.

Many interesting comparisons may be made from the data given in tables 5 and 7. For instance, 2-axle, 6-tire trucks, which are the principal load-carrying single-unit vehicles and which accounted for about 25 percent of

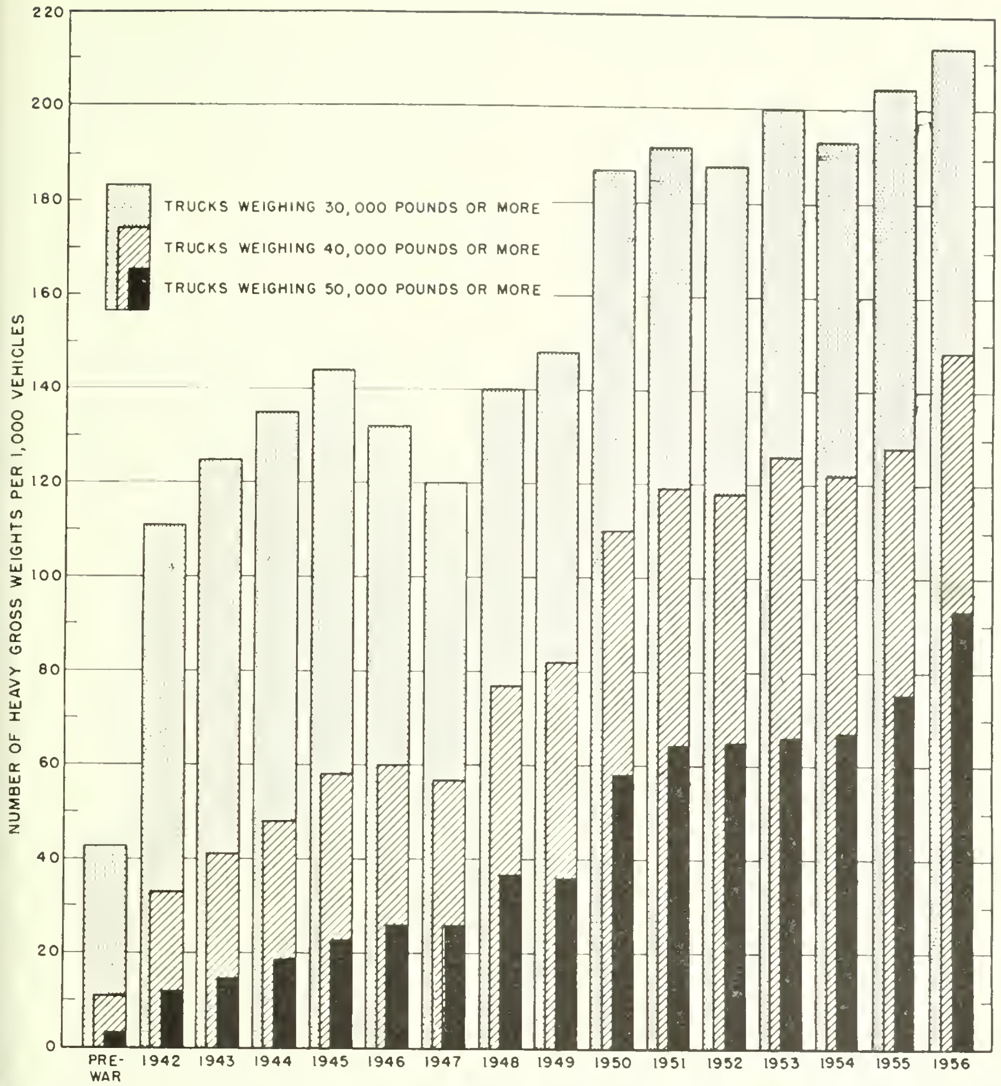


Figure 8.—Number of gross loads of 30,000, 40,000, and 50,000 pounds or more, per 1,000 loaded and empty trucks and truck combinations, on main rural roads in the summers of 1942-56 and a prewar year.

the total truck travel in 1956, carried less than 16 percent of the ton-mileage in that year. On the other hand, 4-axle tractor-semitrailer combinations, an increasingly popular vehicle among truckers, accounted for less than 16 percent of the total truck travel (about two-thirds as much as the 2-axle, 6-tire trucks), but carried almost 40 percent of the ton-mileage or 2½ times the percentage carried by the smaller vehicles.

The actual volume of freight carried annually from 1936 to 1956, inclusive, by trucks and truck combinations on main rural roads is shown in figure 4. The chart illustrates the tremendous growth in ton-miles of freight transported by truck combinations since the beginning of the planning surveys. In 1936, an estimated 13.7 billion ton-miles were transported by combination-type vehicles and slightly less than 14.3 billion ton-miles were transported by single-unit trucks. By 1940, the combination vehicles were hauling slightly more than the single-unit trucks, and in 1956, the ton-mileage hauled by combinations was more than three times that hauled by single-unit trucks.

The growth in ton-mileage by single-unit trucks and truck combinations from 1936 to 1956 is illustrated in another manner in figure 5. This chart shows the changes that have taken place in each of the two factors included in a ton mile. The horizontal scale measures the vehicle-mileage for loaded vehicles of each type, and the vertical scale measures the average carried load. Ton-mileage, the product of these two factors, is represented by the areas of the rectangles.

For single-unit trucks, the increase in ton-mileage from 14.3 billion in 1936 to 41.2 billion in 1956 came about mainly through an increase in the mileage traveled by loaded vehicles, since there was very little increase in the average carried load for this class of vehicle especially from 1951 to 1956. For truck combinations, the increase in ton-mileage from 13.7 billion in 1936 to 130.0 billion in 1956 was the result of a substantial increase in the average carried load and a much greater proportional increase in mileage traveled by loaded vehicles.

A comparison of the estimated percentage of trucks loaded, average carried load, and ton-miles of freight carried on main rural roads in 1936, 1941, 1946, 1951, and 1956 is given separately for single-unit trucks and truck combinations in table 6. The table shows the extent to which the ton-mileage gains were due to increased loading per vehicle. The increases beyond this point resulted from increased travel by loaded vehicles.

In considering the 1956:1936 ratios shown as the final entry of the table, it is observed that the ton-mileage for combinations was almost 10 times as great at the end of this 20-year period as it was at the beginning, while the ratios for the average weight of carried load and the percentage of vehicles loaded were 1.67 and 0.96, respectively. As shown in table 3, the corresponding ratio for the mileage traveled by all vehicles of the combination type, both loaded and empty, was 5.89, and when multiplied by 0.96, the ratio becomes 5.65 for loaded vehicles. Obviously, most of the enormous increase in ton-mileage was due to increased vehicle-mileage rather than to heavier loading, although loading increased 167 percent during the 20-year period.

For single-unit trucks, the percentage of vehicles loaded decreased 16 percent and the average load increased 28 percent during the period 1936–56. The 189 percent increase in ton-mileage for single-unit trucks was therefore only about 13 percent higher than the 168 percent increase in vehicle-mileage, as shown by the ratio in table 3.

The percentage of vehicle-mileage traveled by trucks of various types in 1956 compared with 1951 is shown in figure 6. The trucking industry in recent years has favored the 2-axle truck-tractor with dual-axle semitrailer over other combination types. This was not the case in 1951, when 16.5 percent of all truck travel was performed by 2-axle truck-tractors with single-axle semitrailers, as compared with 10.4 percent by the same type of tractor with dual-axle semitrailers. In 1956, 15.6 percent of all truck travel was by 2-axle truck-tractors with dual-axle semitrailers, whereas the same power unit with single-axle semitrailer accounted for 11.6 percent of the travel. A small increase in travel by 3-axle tractors pulling dual-axle semitrailers likewise may be noted during the period, 1951–56. The shift from 1- to 2-axle semitrailers apparently has taken place in order that maximum possible payloads can be carried under the weight restrictions in effect in the different States.

The percentage of ton-miles hauled by various truck types in 1956 compared with 1951 is shown in figure 7. This chart emphasizes, even more than figure 6, the shift that occurred between 1951 and 1956 from 3-axle tractor-semitrailer combinations to 4- and 5-axle combinations, as measured by the freight carried. In 1956, the 4-axle combinations accounted for more than two and one-third times the ton-mileage carried by the 3-axle combination type, whereas in 1951 the two types carried about the same amount.

Heavy Gross-Load Frequencies Increase

The frequencies of gross loads of 30,000, 40,000, and 50,000 pounds or more, per 1,000 trucks and truck combinations, on main rural roads in the summers of 1942 to 1956, and in a prewar period (1936–37) are shown in figure 8. During this period the trend of the fre-le 8.—Frequency of heavy vehicles of 30,000, 40,000, and 50,000 pounds or more, per 1,000 loaded and empty trucks and truck combinations, on main rural roads in the summer of 1956 by main vehicle types

Vehicle type	Number of vehicles, per 1,000 trucks and truck combinations, weighing—		
	30,000 pounds or more	40,000 pounds or more	50,000 pounds or more
Single-unit trucks:			
2-axle, 6-tire	4	(1)	(1)
3-axle	340	104	16
Average	16	4	1
Truck combinations:			
Truck-tractor and semitrailer	618	438	266
Truck and trailer	716	585	531
Average	625	448	285
Average, all trucks and combinations, 1956	213	148	93
Comparative average, 1955	205	128	75
Comparative average, 1954	193	122	67
Comparative average, 1951	192	119	64
Comparative average, 1946	132	60	26
Comparative average, 1936	43	11	3

[1] Less than 5 per 10,000.

quency of vehicles weighing 50,000 pounds or more was usually upward although at a generally lower rate of increase from 1951 to 1954. The frequency of those weighing 40,000 pounds or more was generally upward with temporary drops in 1947, 1952, and 1954. The long-range trend in frequency of weights of 30,000 pounds or more was also upward, but there was a substantial decline from 1945 to 1947 and temporary declines in 1952 and 1954.

From 1950 to 1954, the frequencies of weights of 30,000 and 40,000 pounds or more have fluctuated, but since 1954 there has been a decided upward trend. The percentage increases in 1956 over 1955 for the 30,000-, 40,000-, and 50,000-pound gross-weight groups were 4, 16, and 24 percent, respectively.

The 1956 gross-weight frequency data, by vehicle types, are presented in table 8. Since no panel and pickups or other 2-axle, 4-tire, single-unit trucks were found in the survey weighing as much as 30,000 pounds, there is no entry for these vehicles in the table. They are included, however, in the total number of vehicles weighed in computing the frequency for all single-unit trucks and for all trucks and combinations.

Table 8 shows that the frequency of heavy vehicles is steadily increasing. In the last 5 years the frequency of vehicles weighing 30,000 pounds or more increased 11 percent, vehicles weighing 40,000 pounds or more increased 24 percent, and those weighing 50,000 pounds or more increased 45 percent. In the last 10 years the increases were 61, 147, and 258 percent, respectively.

Combining the increases in the frequencies of heavy vehicles with the increases in travel by trucks and truck combinations, the mileage traveled in 1956 compared with 1951 by vehicles weighing 30,000, 40,000, and 50,000 pounds or more increased 38, 54, and 80 percent, respectively.

In addition to the gross-weight frequencies of vehicles traveling on main rural roads of the United States, the variation of these frequencies among the several geographical divisions is interesting. The highest frequency in 1956 of vehicles weighing 30,000 pounds or more (264) was found in the East North Central division, with the Pacific division (241) having the second highest frequency. The lowest frequency (176) was in the New England division. Similarly the highest frequency of vehicles weighing 40,000 pounds or more (186) was found in the Pacific division, with the second highest frequency (180) in the East North Central division. At the same time, the highest frequency of vehicles weighing 50,000 pounds or more (163) was found in the Pacific division, with the second highest frequency (112) in the East North Central division. The lowest frequency for this group (56) was found in the New England division. Considering the three weight groups, the frequencies of all heavy loads were greatest in the Pacific division with the East North Central division following closely. On the other hand, the smallest frequencies of such heavy loads were found generally in the New England division.

Heavy Axle-Load Frequencies Gain

The number of axles carrying loads of 18,000, 20,000, and 22,000 pounds or more, per 1,000 trucks and truck combinations, on main rural roads during the period 1942–56, and a prewar year are shown in figure 9. A most important finding indicated in the chart is the reversal in trends that took place in 1951 and 1955. In the first case an upward trend changed to a downward one, and in the second case a second upward trend occurred in all three weight groups. The 1956 frequencies show increases over 1954 and 1955 for each of the three weight groups, and in the case of 18,000- and 22,000-pound axle loads, the 1956 frequencies exceeded those of 1952, while the frequencies of loads weighing 20,000 pounds or more exceeded those of 1953.

The number of axles weighing 18,000 pounds or more in 1950 was more than seven times that in the prewar year, but from the 1950 high there was a drop of 34 percent by 1954.

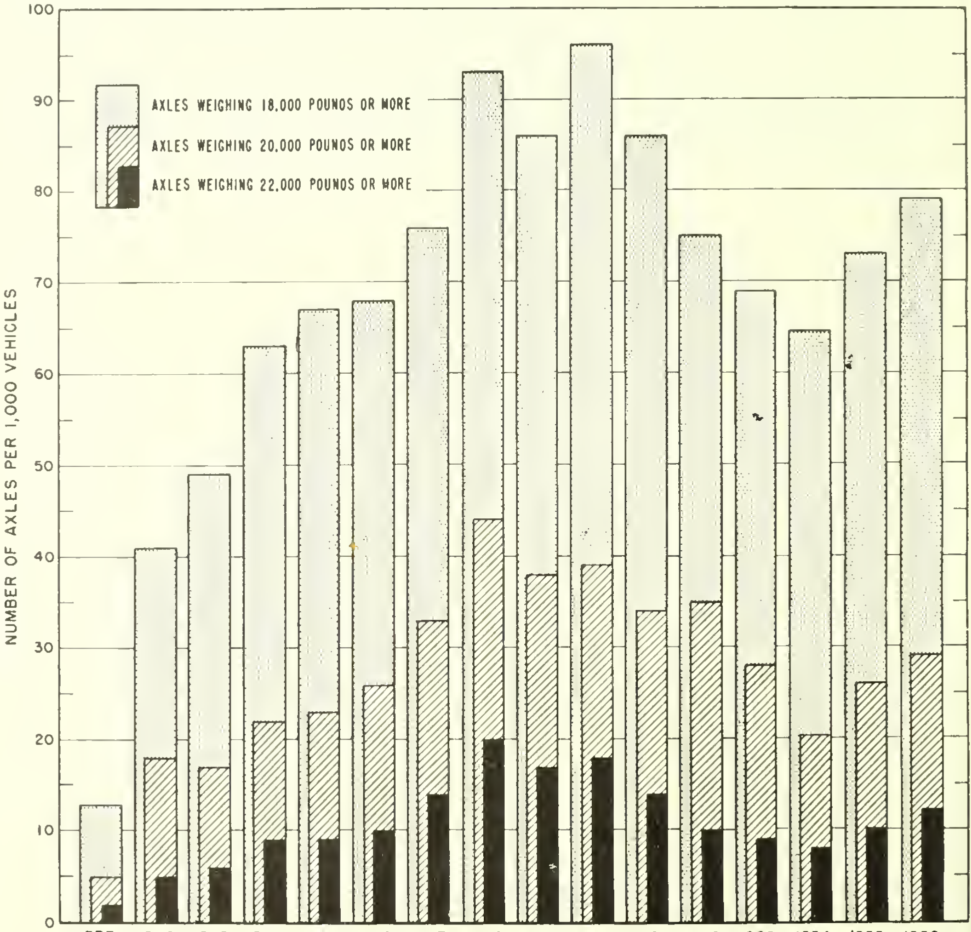


Figure 9.—Number of axles weighing 18,000, 20,000, and 22,000 pounds or more, per 1,000 trucks and truck combinations, on main rural roads in the summers of 1942–56 and a prewar year.

Table 9.—Frequency of axle loads of 18,000, 20,000, and 22,000 pounds or more, per 1,000 loaded and empty trucks and truck combinations, on main rural roads in the summer of 1956 by main vehicle types

Vehicle type	Number of axles, per 1,000 trucks and truck combinations, weighing—		
	18,000 pounds or more	20,000 pounds or more	22,000 pounds or more
Single-unit trucks:			
2-axle, 6-tire	35	12	4
3-axle	95	54	31
Average	17	7	3
Truck combinations:			
Truck-tractor and semitrailer	214	80	33
Truck and trailer	135	17	7
Average	209	76	31
Average, all trucks and combinations, 1956	79	29	12
Comparative average, 1955	73	26	10
Comparative average, 1954	64	20	8
Comparative average, 1951	86	34	14
Comparative average, 1946	68	26	10
Comparative average, 1936	13	5	2

Since 1954, the new upward trend has increased the frequency of this axle-weight group by 23 percent. Likewise, the frequency of 20,000-pound axles in 1950 was almost eight times that in the prewar year, but by 1954 the number in this category was almost one-half of that in 1950. In 1956, the frequency of 20,000-pound loads had increased 45 percent above the 1954 level. The number of axle loads of 22,000 pounds or more increased ninefold from the prewar year to 1950 but dropped 56 percent below that peak in 1954. As in the case of the two lower weight groups, there has been a marked upward trend in 1955 and 1956 for the heaviest axle loads—a 50-percent increase since 1954.

The number of axles weighing 18,000, 20,000, and 22,000 pounds or more for each 1,000 loaded and empty trucks and truck combinations in 1956 are given in table 9 for the main vehicle types. Comparative data are also given for 1955, 1954, 1951, 1946, and 1936. Since none of the 2-axle, single-unit trucks with single rear tires were found to have axles weighing as much as 18,000 pounds, that type is not shown in the table. The number of such vehicles counted are included, however, in obtaining the total frequencies for all single-unit trucks and for all trucks and combinations.

The downward trend in frequencies of heavy axle loads that began in 1950 and extended to 1954 was caused apparently by the shift to vehicles with a larger number of axles. By shifting to tractors and semitrailers with dual axles, truckers are able to haul heavier loads over the highways and yet subject them to less frequent applications of heavy axle loads.

Counteracting the good effects of the shift to vehicles with multiple axles, the increased axle-weight limits, which have been adopted in some States, and the tolerances that have been allowed in many, have made it possible for truckers to increase their payloads without violating the weight laws as they are enforced. These liberalized weight limits apparently are responsible for the increase in frequencies that have occurred in 1955 and 1956.

In order to give a clearer indication of what is happening on the roads, travel by vehicles with axles weighing 18,000, 20,000, and 22,000 pounds or more was calculated in vehicle-miles for the period 1950–56. These calculations show that when vehicle-mileage trends are included, there was an overall upward trend of about 3 percent in axles weighing 18,000 pounds or more, while the travel of vehicles with axles weighing 20,000 and 22,000 pounds or more decreased 7 and 16 percent, respectively. Since axle-load frequencies reached an all-time high in 1950, the above comparisons are made with that year.

Loads Exceeding Legal Limits

The number of trucks and truck combinations in 1956, per 1,000 loaded and empty vehicles, that exceeded the axle, axle-group, or gross-weight limits in effect in the States or recommended by the AASHO, with comparative figures for 1951, are given in table 10. The 1956 frequencies of overloads, especially those including lower percentages of overloads, generally exceeded those for 1951. The 1956 frequencies likewise are somewhat higher than those of the previous year.

This comparison is similar to the variations in heavy axle-load and heavy gross-load frequencies found in the years from 1950 through 1956 as shown in figures 8 and 9. These increased frequencies, which exceed the legal weight limits by small amounts, such as up to 10 percent, are caused in many cases by the tolerances in weight limits (generally 5 percent overloads) that now are being allowed in many States. It appears that in many cases these tolerances are being used as a license to increase the axle-load limit by the amount of the tolerance.

Detailed estimates concerning the overload frequencies in the various geographical areas show that the highest frequency of loads in excess of State legal weight limits in 1956 was

(*Continued on page 267*)

Table 10.—Number of trucks and truck combinations, per 1,000 loaded and empty vehicles, that exceeded any of the axle, axle-group, or gross-weight limits in effect in the States or recommended by the AASHO, in the summer of 1956 and the corresponding comparative figures in 1951

Vehicle type	Vehicles exceeding State legal limits						Vehicles exceeding AASHO recommendations					
	Number per 1,000 overloaded	Number per 1,000 overloaded more than—					Number per 1,000 overloaded	Number per 1,000 overloaded more than—				
		5 percent	10 percent	20 percent	30 percent	50 percent		5 percent	10 percent	20 percent	30 percent	50 percent
VEHICLES OVER LIMITS IN 1956												
2-axle, 6-tire truck	18	9	4	2	1	(1)	30	19	12	5	2	1
3-axle truck	80	49	26	11	5	3	109	71	42	24	13	6
Average, single-unit trucks	10	5	3	1	(1)	(1)	16	10	6	3	1	(1)
Truck-tractor and semitrailer	145	88	48	15	4	1	230	168	111	46	17	3
Truck and trailer	249	83	37	12	6	1	508	369	220	57	25	11
Average, truck combinations	152	87	48	15	5	1	249	182	119	47	17	3
Average, all trucks and combinations	56	32	17	6	2	(1)	91	66	43	17	6	1
VEHICLES OVER LIMITS IN 1951												
2-axle, 6-tire truck	15	10	7	3	2	(1)	21	17	13	7	4	1
3-axle truck	71	46	19	6	3	1	83	57	35	12	5	1
Average, single-unit trucks	9	6	4	2	1	(1)	12	9	7	4	2	(1)
Truck-tractor and semitrailer	139	89	52	20	7	1	198	142	94	40	17	3
Truck and trailer	157	78	32	7	1	(1)	282	211	139	54	10	2
Average, truck combinations	140	88	51	19	7	1	204	147	97	41	17	3
Average, all trucks and combinations	50	32	19	7	3	(1)	72	52	35	16	7	1

[1] Less than 5 per 10,000.

How Access Control Affects Accident Experience

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH BUREAU OF PUBLIC ROADS

Reported by CHARLES W. PRISK,
Director, Highway Safety Study

NEW EVIDENCE has been developed to document the striking safety benefits in the offing for users of the National System of Interstate and Defense Highways now under construction. Full control of access is the key feature in the outlook for safer travel.

Control of access is sometimes regarded as a relatively recent highway refinement. In many ways, however, nearly all of the important design advancements that have gradually evolved to make our highways safer serve also to control access. The grade separation of intersecting highways, for example, is control of access to the extent that the conflicts created by entering and leaving vehicles are eliminated through design of the separation structure and its connecting ramps for interchange traffic. Similarly, the physical division of a highway with a median area introduces access control of a degree because it prohibits or at the very least discourages left-turn access to abutting property.

A few years ago the Bureau of Public Roads invited the cooperation of the State highway departments in a survey of accident experience on controlled access routes. This effort, still continuing, has produced some convincing facts concerning the superior safety associated with control of access.

In most instances, data were obtained for pairs of routes or for abutting sections of highway serving the same general area—one highway with full or partial control of access, and the other a conventional major traffic route without access control. The access control concepts and nomenclature follow the standard definitions approved by the American Association of State Highway Officials. In other words, "full control of access" means that access is provided with selected public roads only and that crossings at grade and direct private driveway connections are prohibited. Where the term "partial control of access" is mentioned, the highway is of a somewhat lower order, and may have some crossings at grade and some private driveway connections.

Table 1 presents the Bureau's most recent compilation of accident experience on highways with full, partial, and no control of access, and was prepared from special reports from 27 States. These data include a total of 47,000 accidents, 30,000 injuries, and nearly 900 deaths occurring on 2,093 miles of highway study sections, and were assembled from records for various years, 1949 through 1955.

Freeways Are Safer

As the mileage figures indicate, composite results are preponderantly weighted by rural sections where fatality rates are commonly higher, and accident rates lower than in urbanized areas. However, for the aggregate experience, drivers on highways with fully planned access are roughly two and one-half times safer from fatalities, injuries, and accidents on a miles-traveled basis than their companions on comparable highways without access control.

The safety characteristics of the fully controlled access highway, which in standard nomenclature is better known as a freeway, are outstanding in the category of rural area fatalities. For example, as table 1 shows, the fatality rate on rural freeways is only one-third that of rural highways without access control, 3.4 deaths per 100 million vehicle-miles in contrast with an average of 10.3 deaths. This finding is heartening indeed when it is remembered that three-fourths of the Nation's traffic deaths now occur outside of urban areas. By contrast, heavy losses in property damage and nonfatal injury accidents are chiefly concentrated in urban and suburban areas. The safety merits of metropolitan area freeways are distinguished more by their markedly lower accident and injury rates, rather than by their improved fatality rates. Full control of access thus offers vital safety advantages for both rural and urban locations.

Table 1.—Accident experience on highways in 27 States with full, partial, and no control of access [1]

Area and degree of access control	Length of study sections	Vehicle-miles of travel	Number of accidents	Number of fatalities	Number of injuries	Rates per 100 million vehicle-miles of travel		
						Accidents	Fatalities	Injuries
	Miles	*Thousands*						
Urban:								
Full control	75.6	1,318,848	2,556	29	1,169	194	2.2	89
Partial control	35.4	573,321	2,997	32	943	523	5.6	164
No control	119.4	1,813,534	9,092	74	4,739	501	4.1	261
Suburban:								
Full control	24.5	198,746	273	8	180	137	4.0	91
Partial control	99.1	613,374	2,253	34	961	367	5.5	157
No control	140.0	1,527,589	7,349	78	5,998	481	5.1	393
Rural:								
Full control	366.5	4,063,863	6,340	137	5,514	156	3.4	136
Partial control	579.0	2,396,668	5,002	151	3,536	209	6.3	148
No control	653.4	3,222,890	11,102	332	6,886	344	10.3	214
Totals:								
Full control	466.6	5,581,457	9,169	174	6,863	164	3.1	123
Partial control	713.5	3,583,363	10,252	217	5,440	286	6.1	152
No control	912.8	6,564,013	27,543	484	17,623	420	7.4	268
Grand total	2,092.9	15,728,833	46,964	875	29,926	298	5.6	190

[1] Data included are for various years, 1949–55, inclusive.

Full and Partial Access Control Compared

The hazards of incomplete treatment of the access problem are interestingly revealed by the accident records included in table 1 for facilities with only partial control. Fatality rates for sections having partial access control are highest on the urban and suburban sections studied, 5.6 and 5.5, respectively. Significant too is the fact that urban and suburban accident and injury rates are roughly double those found on comparable highways with fully planned access. On rural highway sections, the death rate with partial control is nearly twice that found on highways with full control of access.

Another facet of the study concerns th types of accidents that occur. The experience shows that on highways with full control of access, approximately 60 percent o the accidents involve rear-end collision o same-direction sideswipe. On routes wit partial or no access control, the crashes i this category are roughly 40 percent of th total.

Rear-End Collisions, a Problem on Freeways

The higher percentage of rear-end or same-direction collisions on freeways is a matte for serious concern. Checking further, it i found that the rate for rear-end collisions o rural freeways is about 90 per 100 millio vehicle-miles, which is substantially the sam as that for rear-end collisions on other rura highways covered by the study. On urba freeways, however, rear-end collisions occu at an approximate rate of 130 per 100 millio vehicle-miles, and at twice that rate on urba routes without access control. If the freeway has an Achilles heel, it is in its susceptibility, accidentwise, to driving actions tha set up speed differentials, and rear-en collisions. In this connection, the points o entrance and departure are critical ones, and further design refinements will be needed on ingress and egress facilities.

It is heartening to find that on highways with full control of access, fewer than 10 percent of the accidents involve head-on collision, angle collision, and collision with pedestrians. The corresponding values for highway sections with partial or no access control are much higher, ranging from about 35 to 50 percent. This difference is of much importance, for it involves those types of collisions that most frequently cause serious injury or death. The lower incidence of these severe collisions accounts in the main for the low fatality rate.

The world's most renowned highway design, the planned access freeway, clearly sets the scene for superior service to future traffic on the Interstate System. Now that additional mileages of interstate routes are being constructed and opened to traffic, it seems certain that the access control feature will persist as a prime value in highway safety.

Crash-Barrier Tests

(*Continued from page 252*)

nearly 20 feet greater. However, the hedge at the site of test No. 7 was far more dense and the bushes were larger when compared with those for test No. 12. The test car and hedge performance was very similar to that noted in the previous summer tests.

Acknowledgments

The author wishes to acknowledge the splendid cooperation received from personnel of the Bureau of Public Roads, the Connecticut State Highway Department, and the State Police of the Connecticut Motor Vehicle Department.

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Carl C. Saal, of the Bureau of Public Roads, deserves the personal thanks of the author for his assistance during the field tests, and for his careful review and help in preparing this report for publication.

Traffic and Travel Trends

(*Continued from page 265*)

in the Pacific division—78 vehicles in each 1,000 were overloaded to some extent and 2 vehicles in each 1,000 were overloaded by 20 percent or more. The second highest frequency was found in the East South Central division where 71 vehicles in each 1,000 were overloaded and 8 vehicles in each 1,000 were overloaded by 20 percent or more.

A comparison of frequencies of vehicles loaded in excess of the recommendations of the AASHO shows that the highest frequency of heavy loads likewise was in the Pacific division where 179 vehicles in each 1,000 exceeded the recommended weight limits, and 8 in each 1,000 exceeded these limits by 20 percent or more. The second highest frequency was found in the New England division where 117 vehicles in each 1,000 exceeded the recommendations and 56 in each 1,000 exceeded them by 20 percent or more. The high frequencies of loads exceeding the recommendations in the New England division, especially those exceeding the recommendations by 20 percent or more, are caused by the higher legal axle-load limits in effect in that area. Although the maximum axle-load limit recommended by the AASHO and permitted in most States is 18,000 pounds, the legal limit in States of the New England division ranges from 20,000 to 22,400 pounds. Consequently a considerable number of vehicles carrying loads within the State limits may exceed the recommendations of the AASHO by as much as 24 percent.

Motor-Vehicle Size and Weight Limits

A comparison of State legal limits of motor-vehicle sizes and weights with standards recommended by the American Association of State Highway Officials is given in a table on pages 256–257. The statutory limits reported in this tabulation, prepared by the Bureau of Public Roads as of July 1, 1957, have been reviewed for accuracy by the appropriate State officials.

Statutory limits are shown for width, height, and length of vehicles; number of towed units; maximum axle loads for single and tandem axles; and maximum gross weights for single-unit trucks, truck-tractor semitrailer combinations, and other combinations.

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Federal Legislation and Regulations Relating to Highway Construction (1948). Out of print.

Financing of Highways by Counties and Local Rural Governments: 1931–41, 45 cents; 1942–51, 75 cents.

First Progress Report of the Highway Cost Allocation Study, House Document No. 106 (1957). 35 cents.

General Location of the National System of Interstate Highways, Including All Additional Routes at Urban Areas Designated in September 1955. 55 cents.

Highway Bond Calculations (1936). 10 cents.

Highway Bridge Location, No. 1486D (1927). Out of print.

Highway Capacity Manual (1950). $1.00.

Highway Needs of the National Defense, House Document No. 249 (1949). 50 cents.

Highway Practice in the United States of America (1949). 75 cents.

Highway Statistics (annual):
- 1945 (out of print).
- 1946, 50 cents.
- 1947 (out of print).
- 1948, 65 cents.
- 1949, 55 cents.
- 1950 (out of print).
- 1951, 60 cents.
- 1952, 75 cents.
- 1953, $1.00.
- 1954, 75 cents.
- 1955, $1.00.

Highway Statistics, Summary to 1955. $1.00.

Highways in the United States, *nontechnical* (1954). 20 cents.

Highways of History (1939). 25 cents.

Identification of Rock Types (reprint from PUBLIC ROADS, June 1950). 15 cents.

Interregional Highways, House Document No. 379 (1944). 75 cents.

Legal Aspects of Controlling Highway Access (1945). 15 cents.

Local Rural Road Problem (1950). 20 cents.

Manual on Uniform Traffic Control Devices for Streets and Highways (1948) (*including 1954 revisions supplement*). $1.25.

Revisions to the Manual on Uniform Traffic Control Devices for Streets and Highways (1954). *Separate*, 15 cents.

PUBLICATIONS (Continued)

Mathematical Theory of Vibration in Suspension Bridges (1950). $1.25.

Needs of the Highways Systems, 1955–84, House Document No. 120 (1955). 15 cents.

Opportunities in the Bureau of Public Roads for Young Engineers (1955). Out of print.

Parking Guide for Cities (1956). 55 cents.

Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft (1943). $2.00.

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Public Control of Highway Access and Roadside Development (1947). 35 cents.

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Public Utility Relocation Incident to Highway Improvement, House Document No. 127 (1955). 25 cents.

Results of Physical Tests of Road-Building Aggregate (1953). $1.00.

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Selected Bibliography on Highway Finance (1951). 60 cents.

Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways, 1956: a reference guide outline. 55 cents.

Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, FP-57 (1957). $2.00.

Standard Plans for Highway Bridge Superstructures (1956). $1.75.

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MAPS

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United States System of Numbered Highways. 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

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Bibliography on Automobile Parking in the United States (1946).

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF OCTOBER 31, 1957

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			CONTRACTS ADVERTISED, CONSTRUCTION NOT STARTED			PROJECTS UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$38,855	$68,811	$52,766	636.7	$18,218	$13,825	129.9	$72,420	$45,337	685.0	$159,449	$111,928	1,451.6
Arizona	19,792	32,654	28,851	188.1	6,640	5,075	56.8	26,036	22,572	179.5	65,330	56,498	424.4
Arkansas	49,536	27,587	19,360	466.3	15,987	10,958	129.5	48,686	32,525	449.3	92,260	62,843	1,045.1
California	147,690	20,937	14,704	136.3	24,278	17,825	24.8	410,330	216,479	357.2	455,545	249,008	518.3
Colorado	44,736	18,247	13,276	137.7	5,526	4,393	29.3	53,351	38,326	314.7	77,124	55,995	481.7
Connecticut	52,695	2,420	1,570	1.0	6,052	4,031	4.3	40,259	26,911	36.8	48,731	32,512	42.1
Delaware	31,131	2,740	1,398	21.8	2,519	1,233	16.7	13,282	7,680	55.3	18,541	10,311	93.8
Florida	58,220	37,719	25,873	420.0	17,563	13,200	41.6	43,861	25,532	257.6	99,143	64,605	719.2
Georgia	96,008	59,224	37,405	519.8	7,219	4,003	58.2	94,141	54,563	903.1	160,584	95,971	1,481.1
Idaho	41,466	17,946	14,270	105.8	7,463	5,277	43.0	15,921	11,039	144.5	41,330	30,586	293.3
Illinois	92,529	92,032	66,492	484.8	77,435	59,540	132.5	176,899	128,609	735.3	346,366	254,641	1,352.6
Indiana	142,271	27,682	14,946	214.7	16,616	8,599	202.6	51,310	32,212	268.7	95,608	55,757	686.0
Iowa	38,143	69,699	54,711	734.7	11,588	7,245	143.7	62,320	41,952	1,290.3	143,607	103,908	2,168.7
Kansas	49,133	43,082	35,191	786.6	8,178	5,270	101.0	47,056	30,369	1,136.5	98,316	70,830	2,024.1
Kentucky	62,019	38,206	28,385	118.2	7,006	4,153	32.4	57,802	39,535	262.0	103,014	72,073	412.6
Louisiana	67,836	44,233	27,974	267.2	13,291	8,483	75.4	53,266	29,357	338.5	110,790	65,814	681.1
Maine	41,504	7,599	5,247	24.4	1,386	765	7.4	19,132	10,433	115.1	28,117	16,445	146.9
Maryland	15,199	38,346	24,065	91.0	22,433	16,276	24.4	63,504	43,769	181.2	124,283	84,110	296.6
Massachusetts	55,513	50,737	38,077	33.3	52,879	33,115	36.6	66,777	40,623	46.5	170,393	111,815	116.4
Michigan	101,523	79,319	59,173	446.7	33,941	20,171	38.1	102,490	71,314	580.4	215,750	150,658	1,065.2
Minnesota	66,742	7,867	6,793	70.3	3,042	2,609	.5	112,728	82,292	1,534.1	123,637	91,694	1,604.9
Mississippi	20,426	52,048	39,245	700.7	23,790	19,772	100.0	58.008	38,341	850.8	133,846	97,358	1,651.5
Missouri	75,072	44,912	28,931	999.9	12,471	9,903	16.8	116,461	74,964	1,250.3	173,844	113,798	2,267.0
Montana	66,609	14,111	10,161	267.4	4,747	3,122	33.9	41,090	30,602	250.5	59,948	43,885	551.8
Nebraska	78,848	11,164	5,914	213.2	9,881	5,740	83.2	40,762	24,091	1,156.7	61,807	35,745	1,453.1
Nevada	32,677	13,416	12,596	29.0				27,461	24,847	130.0	40,877	37,443	159.0
New Hampshire	19,473	9,333	7,213	21.8	2,443	1,770	4.7	21,402	14,264	62.6	33,178	23,247	89.1
New Jersey	102,670	10,119	5,563	71.6	6,808	3,655	2.0	75,327	53,085	57.6	92,254	62,303	131.2
New Mexico	29,802	7,009	4,280	76.2	9,995	7,737	71.8	36,240	30,019	252.6	53,244	42,036	400.6
New York	168,564	35,253	22,116	92.9	105,067	75,798	75.1	488,605	293,652	495.4	628,925	391,566	663.4
North Carolina	103,900	25,093	16,313	213.3	10,327	7,516	36.0	81,423	50,466	875.0	116,843	74,295	1,124.3
North Dakota	39,688	19,091	13,947	860.0	5,236	4,211	59.4	22,229	14,377	997.3	46,556	32,535	1,916.7
Ohio	40,378	148,724	107,298	249.9	31,760	25,597	33.1	210,861	142,345	303.4	391,345	275,240	586.4
Oklahoma	46,255	56,280	41,739	499.6	12,988	9,304	68.0	61,134	37,861	569.5	130,402	88,904	1,137.1
Oregon	41,126	8,381	6,518	28.2	8,928	6,863	35.0	43,904	33,554	213.9	61,213	46,935	277.1
Pennsylvania	156,705	86,462	55,661	200.9	49,304	37,122	73.8	234,675	150,298	372.8	370,441	243,081	647.5
Rhode Island	17,606	4,584	2,405	6.4	3,320	2,848	1.6	37,747	25,223	23.9	45,651	30,476	31.9
South Carolina	49,869	43,557	30,684	454.3	3,051	2,463	17.5	38,053	22,256	746.3	84,661	55,403	1,218.1
South Dakota	40,979	27,554	21,668	341.9	553	310	24.4	22,324	15,783	492.7	50,431	37,761	859.0
Tennessee	55,555	68,291	51,612	354.6	14,597	9,157	22.5	84,662	51,280	520.8	167,550	112,049	897.9
Texas	210,119	18,159	9,909	446,7	66,230	51,872	210.2	173,399	116,175	1,553.6	257,788	177,956	2,210.5
Utah	9,110	38,527	34,800	220.8	4,177	3,117	26.2	23,900	19,668	105.8	66,604	57,585	352.8
Vermont	23,526	3,941	2,143	32.0	47	23	.2	20,464	15,219	39.8	24,452	17,385	72.0
Virginia	29,416	67,139	54,626	134.6	22,530	16,974	46.4	62,033	41,149	263.7	151,702	112,749	444.7
Washington	58,094	21,148	15,686	113.6	4,528	3,850	41.8	54,527	39,123	253.6	80,203	58,659	409.0
West Virginia	42,389	45,913	34,177	82.1	4,801	2,425	11.4	38,853	21,375	84.6	89,567	57,977	178.1
Wisconsin	102,601	17,835	10,838	272.8	11,368	8,377	87.1	57,717	35,552	394.9	86,920	54,767	754.8
Wyoming	5,460	25,853	21,071	178.7	4,102	2,825	23.3	49,491	40,843	330.5	79,446	64,739	532.5
Hawaii	5,858	8,583	4,291	13.4	1,272	631	3.6	2,677	1,256	1.3	12,532	6,178	18.3
District of Columbia	29,395	7,290	5,274	3.3	4,649	3,174	2.9	17,598	14,197	3.2	29,537	22,645	9.4
Puerto Rico	13,985	9,160	4,100	7.9				18,991	9,005	57.0	28,151	13,105	64.9
Alaska	13,570	4,130	4,130	58.8	1,087	1,087	23.2	10,594	9,087	335.0	15,811	14,304	417.0
TOTAL	3,042,266	1,740,147	1,255,436	13,151.9	799,317	573,289	2,563.8	3,904,183	2,521,386	22,916.7	6,443,647	4,350,111	38,632.4

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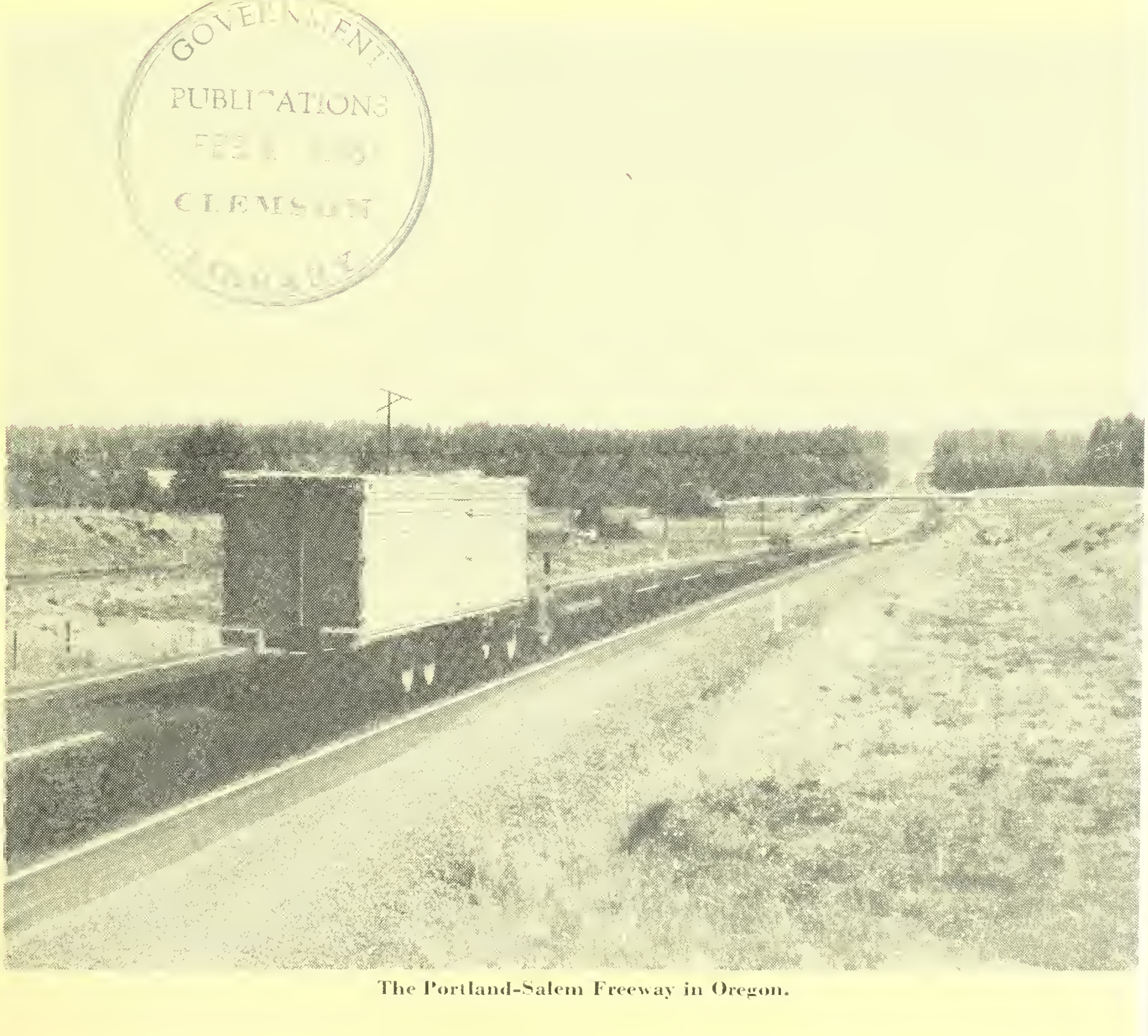

The Portland-Salem Freeway in Oregon.

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U. S. DEPARTMENT OF COMMERCE
SINCLAIR WEEKS, Secretary

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BERTRAM D. TALLAMY, Administrator

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A Survey of Air Pressures of Tires Mounted on Trucks Operating in the Everyday Traffic

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH
BUREAU OF PUBLIC ROADS

Reported by CARL C. SAAL, Chief,
Vehicle Operations Branch

This article discusses the results of a survey made in 1954 of tire pressures of trucks operating in the general traffic stream. Tire sizes included in the analysis ranged from 7.00 x 18, 8 ply, usually found on the lightweight single-unit trucks, to 11.00 x 24, 12 ply, used for the most part on the heaviest truck combinations. Inflation pressures and corresponding wheel loads were determined for about 8,000 vehicles and over 40,000 tires.

Over-the-road inflation pressures of dual tires of the smaller sizes were found to be about equal to or less than the cold inflation pressures recommended by the Tire and Rim Association, Inc., as revised on April 15, 1955. For tire sizes ranging from 9.00 x 20, 10 ply, and upward, the average hot inflation pressures were about 10 percent above the recommended pressures.

Air pressures of tires mounted on dual wheels did not vary significantly because of axle position or when mounted on the inside or outside of a dual-wheel assembly. Average inflation values for single tires on the front axle of vehicles were slightly less than the pressures of dual tires mounted on the other axles.

In most cases the average wheel load for dual tires was less than 70 percent of the loads recommended by the Tire and Rim Association, Inc., as revised on April 15, 1955. The increase in pressure was found to be very gradual over the entire load range, and in general amounted to slightly more than 10 percent from the lowest to the highest tire loads.

Ambient temperatures ranging from 60 to 95 degrees F. caused only small increases in the inflation pressures of tires checked on the highways. Above 95 degrees, rather sharp increases in tire pressures were found in some instances.

In recent years, there has been some tendency for truck operators to use high-pressure tires. These so-called "high-load" tires are constructed with all-nylon cord or with steel cord. Only a limited number of these tires were checked in this survey, but from the small sample it was found that hot inflation pressures averaged 9 pounds per square inch in excess of those for the comparable conventional tires.

A more recent survey of motor-carrier practices, concerning their tire inflation specifications for fleet operation, revealed that for three common tire sizes, the specifications exceeded the recommendations of the Tire and Rim Association, Inc., for cold inflation pressures by about 5 pounds per square inch.

THE inflation pressure of tires, as they run "hot" in service on trucks and truck combination units, was investigated for immediate use in planning and operating the AASHO Road Test Project at Ottawa, Ill. It is also expected that highway designers in general will be interested in the results.

The study was conducted in 37 States in conjunction with the annual summer loadometer surveys in 1954. In each of these States at least one station at which a large volume of truck traffic could be expected was selected and operated. The tire pressures and corresponding wheel loads were measured on the right-hand side of about 8,000 vehicles, involving some 40,000 tires.

The principal objectives of this study were to determine for given tire sizes the frequency of inflation pressures and the relation between wheel load and inflation pressure. In addition to the tire pressure results for vehicles in normal operation, the report describes some current practices of truck operators in inflating cold tires and investigates the trend in tire inflation practices to a limited extent by comparing the present results with those reported by Kansas in 1941.

Summary of Findings

1. In the case of dual tires, the "hot" inflation pressures on the average were either approximately equal to or less than the revised recommended "cold" inflation pressures of the Tire and Rim Association, Inc., for tire sizes of 8.25 x 20, 12 ply, and smaller. For tire sizes of 9.00 x 20, 10 ply, and larger, the average hot inflation pressures were about 10 percent above the recommended ones.

2. The average hot inflation pressures of dual tires lie between 80 and 85 pounds per square inch for tire sizes ranging from 10.00 x 20, 12 ply, to 11.00 x 24, 12 ply. The corresponding range of 85-percentile values was 90 to 95 p.s.i. The revised recommended cold pressure for this group of tire sizes was 75 p.s.i.

3. An analysis of average tire pressures for a given tire size and vehicle type indicated that it was only necessary to group the data by the first (front) axle and by the remaining axles with dual tires. The position of the dual tire with respect to axle configuration was determined not to be a factor.

4. The average pressure for single tires on the first axle was always slightly less than that for dual tires on the other axles. Also, on the average, the variation in pressure between the inside and outside tires mounted on dual wheels was found to be very small.

5. The average tire load for dual tires of a given size was generally less than 70 percent of the revised recommended load. Also, the 85-percentile values did not exceed the revised schedule of loads except for the 7.50 x 20, 10-ply and the two 8.25 x 20 tire sizes, and then only to a small degree.

6. The increase of average hot inflation pressures over the tire load range was very gradual and generally amounted to slightly over 10 percent from the lowest to the highest tire load. The increase was very insignificant for the range of load for a given tire size that would be considered in highway design.

7. The average hot inflation pressures did not vary materially with vehicle type or geographical region. On the basis of the results of this survey, it is reasonable to assume that the average values for the country as a whole and for all vehicle types may be applied universally.

8. For a given tire size, the average hot inflation pressure did not appear to increase appreciably with an increase in ambient temperature between 60° and 95° F. Above 95° there was a tendency for the pressure to increase sharply in some instances.

9. On the basis of very limited data, it appears that the so-called "high-load" tire might have average hot pressures of at least

Table 1.—Number of vehicles included in tire-pressure survey, classified by vehicle type and grouped by regions

Vehicle type [1]	Region 1 [2]			Region 2 [4]			Region 3			Region 4 [5]			All regions		
	Number of vehicles	Number of tires [3]	Number of tires checked	Number of vehicles	Number of tires [3]	Number of tires checked	Number of vehicles	Number of tires [3]	Number of tires checked	Number of vehicles	Number of tires [3]	Number of tires checked	Number of vehicles	Number of tires [3]	Number of tires checked
Single-unit trucks:															
2D	243	729	699	780	2, 340	2, 170	869	2, 607	2, 528	348	1, 044	944	2, 240	6, 720	6, 341
3	30	150	143	75	375	348	58	290	273	66	330	241	229	1, 145	1, 005
Tractor-semitrailer combinations:															
2-S1	350	1, 750	1, 694	871	4, 355	4, 139	711	3, 555	3, 366	120	600	521	2, 052	10, 260	9, 720
2-S2	211	1, 477	1, 266	1, 152	8, 064	7, 577	1, 143	8, 001	7, 655	173	1, 211	1, 031	2, 679	18, 753	17, 529
3-S2	2	18	18	7	63	58	255	2, 295	2, 112	252	2, 268	1, 848	516	4, 644	4, 036
Tractor-semitrailer and trailer combinations:															
2-S1-2	------	------	------	------	------	------	23	207	196	42	378	236	65	585	432
2-S2-2	------	------	------	------	------	------	17	187	171	1	11	6	18	198	177
Truck and trailer combinations:															
2-2	2	14	14	------	------	------	9	63	55	6	42	33	17	119	102
3-2	------	------	------	------	------	------	8	72	67	71	639	508	79	711	575
3-3	------	------	------	------	------	------	------	------	------	31	341	257	31	341	257
Total trucks and combinations	838	4, 138	3, 834	2, 885	15, 197	14, 292	3, 093	17, 277	16, 423	1, 110	6, 864	5, 625	7, 926	43, 476	40, 174

[1] For an explanation of the vehicle code, see text in right-hand column.
[2] No data for Maine, New Hampshire, New York, Rhode Island, Vermont, and District of Columbia.
[3] Only the tires on the right side of vehicles.
[4] No data for Florida, Tennessee, and West Virginia.
[5] No data for Montana, Texas, and Utah.

Table 2.—Frequency of tires by size and axle position

Tire size	Number of ply	Number of tires by position				
		First axle	Other axles with dual tires			Total tires, all axles
			Inside tires	Outside tires	Total tires	
7.00 x 18	8	61	53	59	112	173
7.00 x 20	8	72	50	54	104	176
7.00 x 20	10	41	28	33	61	102
7.50 x 20	8	479	194	221	415	894
7.50 x 20	10	359	252	276	528	887
8.25 x 20	10	1, 327	1, 645	1, 911	3, 556	4, 883
8.25 x 20	12	59	107	137	244	303
9.00 x 20	10	857	1, 432	1, 622	3, 054	3, 911
10.00 x 20	12	2, 577	6, 683	7, 567	14, 250	16, 827
10.00 x 22	12	700	1, 906	2. 350	4, 256	4, 956
11.00 x 20	12	261	675	776	1, 451	1, 712
11.00 x 22	12	326	787	952	1, 739	2, 065
11.00 x 24	12	10	18	18	36	46
Total tires	------	7, 129	13, 830	15, 976	29, 806	36, 935

10 p. s. i. in excess of those found in comparable conventional sizes, but not to the extent of the pressure differential between the recommended cold inflation pressures.

10. The cold inflation pressures specified by motor carriers for their fleets were obtained in a recent survey of truck operation practices and were found for three common tire sizes to be about 5 p.s.i., on the average, above the recommended pressure.

11. The tire pressures measured on a similar survey made in Kansas about 1940 compared very closely with those obtained by the present survey for three tire sizes. For two smaller tire sizes (7.50 and 7.00), the 1954 values were appreciably higher.

12. Gaged by available reference data from controlled tests, the hot inflation pressure levels established by this survey appear to be well within the range that could be expected by following the recommended practice of inflating cold tires.

Table 3.—Average air pressure for 10.00 x 20, 12-ply tires checked on truck-tractor-semitrailer combinations in region 3

Axle position	Number of tires and average pressure for 3 vehicle types					
	2-S1		2-S2		3-S2	
	Number of tires	Air pressure	Number of tires	Air pressure	Number of tires	Air pressure
		P. s. i.		*P. s. i.*		*P. s. i.*
First	191	73. 1	848	73. 7	103	77. 6
Second	401	79. 5	1, 628	80. 4	161	79. 5
Third	486	79. 4	1, 702	80. 7	163	80. 2
Fourth	------	------	1, 686	81. 9	180	79. 6
Fifth	------	------	------	-------	175	79. 5

Vehicle Type Code

In some instances vehicle types are represented by the general code in common use. Each digit of the code indicates the number of axles of a vehicle or of a unit of a vehicle combination. A single digit, or the first digit of a group symbol, represents a single-unit truck or, if followed by an "S," a truck-tractor. The "S" designation, of course, represents a semitrailer. A digit without an "S" in the second or third position of a group symbol represents a full trailer.

2D =2-axle single-unit truck with dual rear tires
3 =3-axle single-unit truck
2-S1 =2-axle truck-tractor with 1-axle semitrailer
2-S2 =2-axle truck-tractor with 2-axle semitrailer
3-S2 =3-axle truck-tractor with 2-axle semitrailer
2-2 =2-axle truck with 2-axle trailer
3-2 =3-axle truck with 2-axle trailer
3-3 =3-axle truck with 3-axle trailer
2-S1-2=2-axle truck-tractor with 1-axle semitrailer and 2-axle trailer
2-S2-2=2-axle truck-tractor with 2-axle semitrailer and 2-axle trailer

Procedure

The field data were recorded on a form which provided space for indicating vehicle type, tire size, tire pressure for each tire on the right-hand side of vehicles, and wheel or axle load depending on whether wheel loadometers or pit scales were used. Other information entered on the form included station number, the date, hour of operation, the ambient temperature, and the condition of the pavement surface (wet or dry).

Generally, two men in addition to the normal crew were required, one to obtain the tire sizes and inflation pressures, and another to record these data and to obtain the axle weight or wheel weights from the weighmaster. Tire pressures measured with an accurate truck-type tire gage were usually recorded to the nearest 5 pounds.

Vehicles were not delayed excessively for any reason. For instance, no attempt was made to gage the pressure if the valve stem was not readily accessible. A seemingly small

Table 4.—Summary of average air pressure of tires, according to size and position on vehicle

Tire size	Number of ply	Average tire pressure by position					Maximum recommended tire pressure[1]	
		First axle	Other axles with dual tires			All axles	Before Apr. 15, 1955	After Apr. 15, 1955
			Inside tires	Outside tires	All tires			
		P. s. i.	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*
7.00 x 18	8	49	50	52	51	50	55	60
7.00 x 20	8	55	57	58	57	56	55	60
7.00 x 20	10	54	62	61	62	59	70	75
7.50 x 20	8	56	62	62	62	59	60	65
7.50 x 20	10	59	68	67	67	64	70	75
8.25 x 20	10	63	72	72	72	69	65	70
8.25 x 20	12	65	75	72	73	71	75	80
9.00 x 20	10	68	76	76	76	74	65	70
10.00 x 20	12	73	81	81	81	80	70	75
10.00 x 22	12	80	81	81	81	81	70	75
11.00 x 20	12	76	82	82	82	81	70	75
11.00 x 22	12	78	82	82	82	81	70	75
11.00 x 24	12	79	84	83	84	83	70	75

[1] Tire and Rim Association, Inc., Year Book.

Table 5.—Comparison of average observed air pressure of tires on first axle and other axles with maximum recommended air pressure

Tire size	Number of ply	Ratio of average observed tire pressure to maximum recommended pressure—			
		Before Apr. 15, 1955		After Apr. 15, 1955	
		First axle	Other axles	First axle	Other axles
7.00 x 18	8	0.89	0.93	0.82	0.85
7.00 x 20	8	1.00	1.04	.92	.95
7.00 x 20	10	.77	.88	.72	.83
7.50 x 20	8	.93	1.03	.86	.95
7.50 x 20	10	.84	.96	.79	.89
8.25 x 20	10	.97	1.11	.90	1.03
8.25 x 20	12	.86	.97	.81	.91
9.00 x 20	10	1.05	1.17	.97	1.09
10.00 x 20	12	1.04	1.16	.97	1.08
10.00 x 22	12	1.14	1.16	1.07	1.08
11.00 x 20	12	1.09	1.17	1.01	1.09
11.00 x 22	12	1.12	1.17	1.04	1.09

but very important item was the necessity to exercise extreme care in closing all valve cores. At least one State required the driver to sign a statement to the effect that the State was relieved from any responsibility in the case of subsequent tire failures.

The field data were first grouped by the four AASHO regions[1] and by vehicle types. A classification of the sample by region and vehicle type, including the number of tires involved, is shown in table 1. The size of the samples ranged from 54 vehicles in Idaho to 640 in Arkansas.

The next step was to group the data by tire size. The total sample, considering ply, included data for 103 tire sizes. The sizes found most frequently are shown in table 2. With the exception of the 11.00 x 24, 12-ply tire size, in which case the sample was very small, these are the sizes that are considered throughout the analysis. The average data for the 11.00 x 24, 12-ply tire size are included in the general results because of the interest in the operation of large tire sizes.

Another factor that had to be considered was the position of the tire with respect to axle configuration. An analysis of tire pressures for a given size indicated that it was necessary only to group the pressures by the first axle and by the remaining axles with dual tires. For example, the results of this analysis for 10.00 x 20, 12-ply tires in region 3 are summarized in table 3. It should be noted that the average pressures for the axles with dual tires are shown to be in close agreement for the three vehicle types. The distribution of the sample by tire size for the adopted axle grouping is shown in table 2.

Considering the fact that tire pressure is a function of load, miles operated without stops, speed, and surface type, as well as ambient and/or road surface temperature, and that control of all these variables was impractical if not impossible, it was not ex-

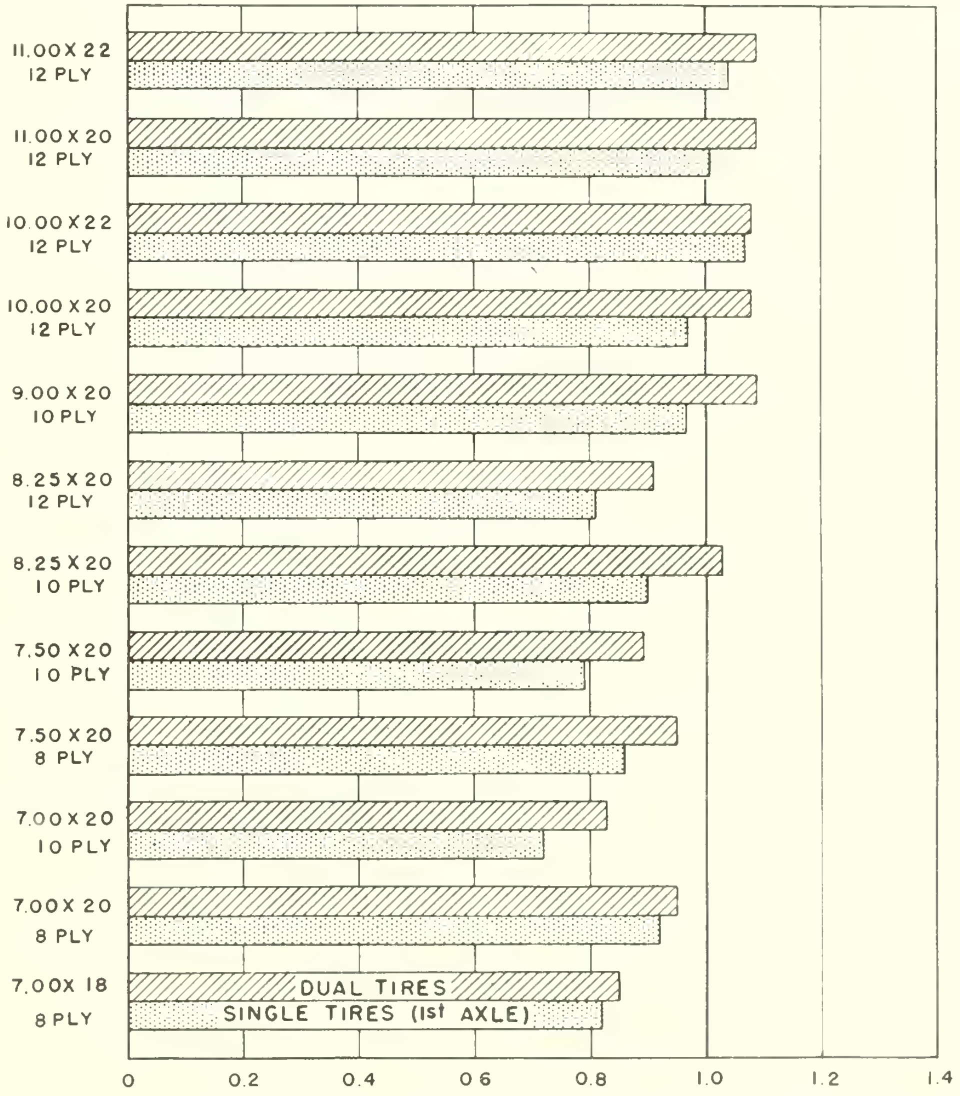

Figure 1.—Average observed tire pressure compared with pressure recommended by the Tire and Rim Association, Inc., after April 15, 1955.

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[1] *Region 1.*—Connecticut, Delaware, Maine, Maryland, Massachusetts, New Hampshire, New Jersey, New York, Pennsylvania, Rhode Island, Vermont, and District of Columbia. *Region 2.*—Alabama, Arkansas, Florida, Georgia, Kentucky, Louisiana, Mississippi, North Carolina, South Carolina, Tennessee, Virginia, and West Virginia. *Region 3.*—Illinois, Indiana, Iowa, Kansas, Michigan, Minnesota, Missouri, Nebraska, North Dakota, Ohio, Oklahoma, South Dakota, and Wisconsin. *Region 4.*—Arizona, California, Colorado, Idaho, Montana, Nevada, New Mexico, Oregon, Texas, Utah, Washington, and Wyoming.

Table 6.—Comparison of 15-, 50-, and 85-percentile tire pressures of dual tires with maximum recommended pressure

Tire size	Number of ply	Ratio of observed tire pressure to maximum recommended pressure—					
		Before Apr. 15, 1955			After Apr. 15, 1955		
		15 percentile	50 percentile	85 percentile	15 percentile	50 percentile	85 percentile
7.00 x 18	8	0.76	0.98	1.13	0.70	0.88	1.08
7.00 x 20	8	.86	1.09	1.31	.79	1.00	1.20
7.00 x 20	10	.76	.92	1.06	.71	.86	.99
7.50 x 20	8	.85	1.08	1.28	.78	1.00	1.18
7.50 x 20	10	.80	1.01	1.19	.75	.94	1.11
8.25 x 20	10	.95	1.15	1.34	.88	1.06	1.24
8.25 x 20	12	.85	1.03	1.16	.80	.96	1.09
9.00 x 20	10	1.02	1.20	1.38	.95	1.13	1.28
10.00 x 20	12	1.04	1.19	1.34	.97	1.11	1.25
10.00 x 22	12	1.04	1.19	1.34	.97	1.11	1.25
11.00 x 20	12	1.05	1.21	1.36	.98	1.13	1.27
11.00 x 22	12	1.07	1.21	1.36	1.00	1.13	1.27

pected that the survey would clearly define the effect of ambient temperature on tire-pressure levels. However, examples are given later in the report to indicate how pressure varied with ambient temperature.

The average temperatures for the four regions in numerical order were 79, 84, 80, and 72 degrees, respectively. These averages were obtained by averaging the hourly temperatures for the period of study. It is evident that the survey was conducted during the more critical period of the year with respect to tire inflation pressures. The temperatures ranged from a minimum of 40 degrees in Nevada to a maximum of 108 degrees in Missouri.

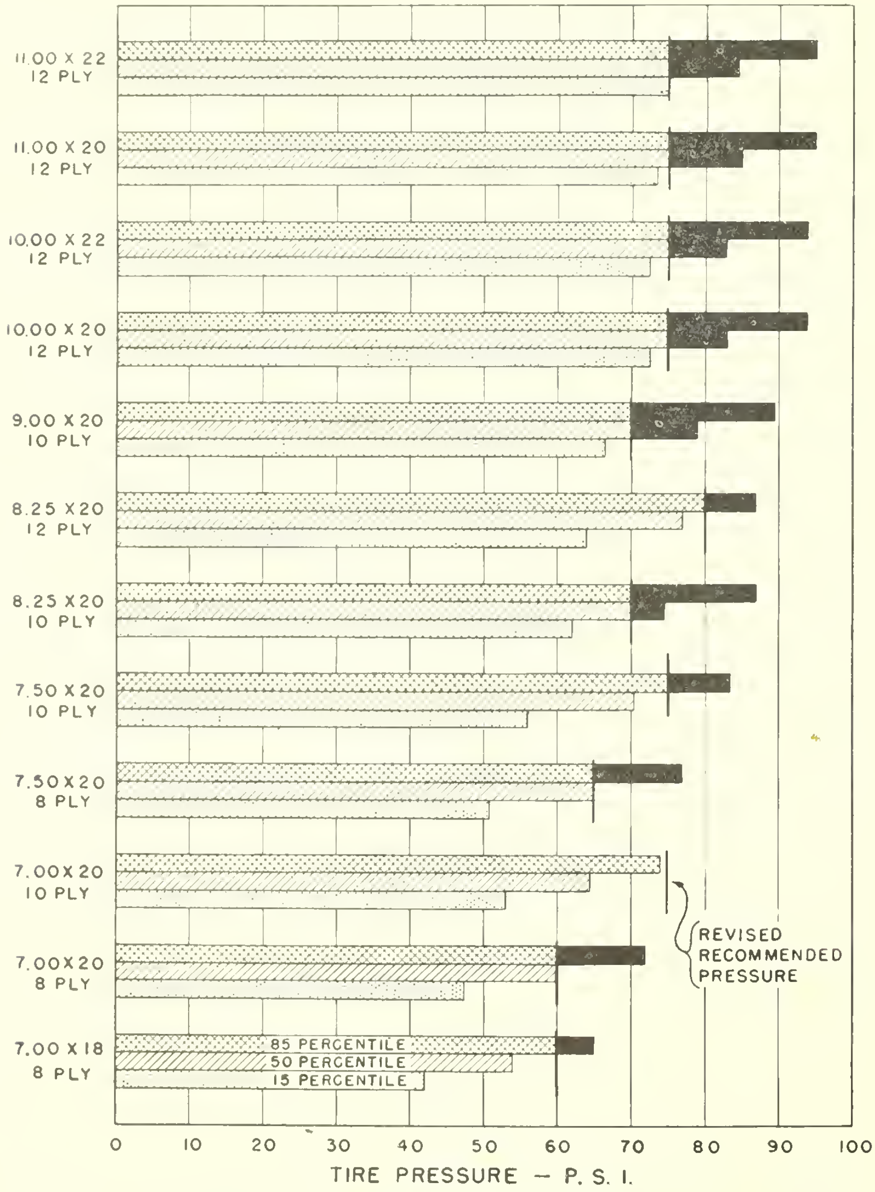

Figure 2.—Comparison of 15-, 50-, and 85-percentile values of tire pressures for dual tires.

Tire-Pressure Observations

The average results of the tire-pressure survey are summarized for the common tire sizes in table 4. In this table are shown the maximum recommended tire pressures of the Tire and Rim Association, Inc., and the average tire pressures observed for selected axle and wheel configurations. The average pressure for the tires on the first axle was always slightly less than that for the other axles with dual tires. Another point of interest is the small variation in average pressure between the inside and outside tires mounted on dual wheels. Average hot inflation pressures for the dual tires ranged between 80 and 85 p.s.i. for tire sizes of 10.00 x 20, 12 ply and greater.

Two schedules of recommended pressures are shown in table 4. This was necessary since the Tire and Rim Association, Inc., revised their recommendations on April 15, 1955, after the period of the tire survey. The increase in the recommended values is not believed to have altered materially the practice of inflating tires, because the new schedule only endorsed what had been the rule. The effect, if any, would be to raise the level of tire pressures established by the 1954 survey. In some of the later presentations of data, both schedules are used as bases for comparisons.

The extent to which the average pressure varies from that recommended for a given tire size is indicated by the pressure ratios in table 5. For example, if the ratio of observed pressure to recommended pressure was 1.08, then the observed value was about 8 percent greater than the recommended value. The ratios computed on the basis of the new schedule of recommended pressures are shown in figure 1. It is seen that dual tires (other axles) had an average pressure greater than the recommended pressure for the 8.25 x 20, 10-ply size and for the 9.00 x 20, 10-ply and greater sizes. In contrast, the single tires on the first axle had an average pressure greater than the recommended pressure only for the three largest tire sizes. In the case of dual tires, the 9.00 x 20, 10-ply and greater tire sizes were inflated on the average almost 10 percent above the new schedule, and more than 15 percent above the old schedule of recommended pressures.

The frequency distribution of tire pressures for a given tire size is indicated partially in figure 2. It is thought that the 85-percentile values are the most pertinent from the viewpoint of highway design. Those values range from 65 p.s.i. for 7.00 x 18 tires to 95 p.s.i. for the two 11.00 size tires. For tire sizes of 8.25

Table 7.—Summary of average tire loads by tire size and position on vehicle

Tire size	Number of ply	Average tire load by position			Maximum recommended tire load [2]	
		First axle	Other axles [1]	All axles	Before Apr. 15, 1955	After Apr. 15, 1955
		Lbs.	*Lbs.*	*Lbs.*	*Lbs.*	*Lbs.*
7.00 x 18	8	1, 510	1, 280	1, 380	1, 850	2, 140
7.00 x 20	8	1, 690	1, 550	1, 600	2, 000	2, 310
7.00 x 20	10	1, 760	1, 645	1, 690	2, 250	2, 630
7.50 x 20	8	1, 775	1, 765	1, 770	2, 375	2, 740
7.50 x 20	10	1, 760	1, 650	1, 690	2, 700	2, 980
8.25 x 20	10	2, 070	2, 415	2, 320	2, 900	3, 330
8.25 x 20	12	2, 425	2, 300	2, 325	3, 150	3, 600
9.00 x 20	10	2, 540	2, 705	2, 670	3, 450	3, 960
10.00 x 20	12	3, 095	2, 870	2, 910	4, 000	4, 580
10.00 x 22	12	3, 430	2, 805	2, 965	4, 275	4, 880
11.00 x 20	12	3, 530	3, 115	3, 180	4, 500	5, 150
11.00 x 22	12	3, 955	3, 170	3, 295	4, 750	5, 480

[1] To determine load on a single tire of a dual assembly, the wheel load was divided by 2, or the axle load was divided by 4.

[2] Tire and Rim Association, Inc., Year Book.

and greater the 85-percentile pressures are above 85 p.s.i. The spread between the 15- and 85-percentile values is confined within the range of 20 to 27 p.s.i. Figure 2 shows that tire inflation pressures of 90 p.s.i. are rather common for commercial vehicles operating in the general traffic.

The tire-pressure ratios, similar to those shown in table 5 for average inflation pressures, are given in table 6 for the 15-, 50-, and 85-percentile tire pressures plotted in figure 2. For the larger tire sizes, the 85-percentile inflation pressures exceed the new schedule of recommended pressures by about 25 percent.

Tire-Load Observations

An analysis of tire loads was made in the same manner as that just described for tire pressures. It is not intended that the results should be used for design or other purposes, as much more quantitative data are available from the more comprehensive regular loadometer surveys. The purpose is merely to give a general picture of the range of tire loads found in the tire-pressure survey.

Table 8.—Comparison of average observed tire loads on first axle and other axles with maximum recommended load

Tire size	Number of ply	Ratio of average observed load to maximum recommended load—			
		Before Apr. 15, 1955		After Apr. 15, 1955	
		First axle	Other axles	First axle	Other axles
7.00 x 18	8	0. 82	0. 69	0. 71	0. 60
7.00 x 20	8	. 84	. 77	. 73	. 67
7.00 x 20	10	. 78	. 73	. 67	. 63
7.50 x 20	8	. 75	. 74	. 65	. 65
7.50 x 20	10	. 65	. 61	. 59	. 56
8.25 x 20	10	. 71	. 83	. 62	. 72
8.25 x 20	12	. 77	. 73	. 67	. 64
9.00 x 20	10	. 74	. 78	. 64	. 68
10.00 x 20	12	. 77	. 72	. 67	. 63
10.00 x 22	12	. 80	. 66	. 70	. 57
11.00 x 20	12	. 78	. 69	. 68	. 60
11.00 x 22	12	. 83	. 67	. 72	. 58

Table 7 summarizes the average tire loads for each tire size by axle configuration. Also included in the table for reference purposes are the two schedules of recommended tire loads which are compatible with the recommended tire pressures. It is evident that the average tire loads are well below those recommended by the Tire and Rim Association, Inc. The extent of this variation is shown in table 8 which contains the ratios of observed to recommended loads. It is seen that tires on the front axle were loaded heavier than those on the other axles in all except two instances. The average tire load was generally less than 70 percent of the new load schedule.

The 15-, 50-, and 85-percentile tire loads for the dual-tired wheels (other axles) are plotted in figure 3 for the several tire sizes. There was a wide range of tire loads for a given tire size especially for the larger sizes. Also, the 85-percentile values did not exceed the new schedule of loads except for the 7.50 x 20, 10-ply and the two 8.25 x 20 tire sizes, and then only to a small degree.

The ratios of the tire loads shown in figure 3 to both the old and new schedules are listed in table 9 for the respective tire sizes. A study of the ratios in table 9 definitely reveals that present operating practices result in tire

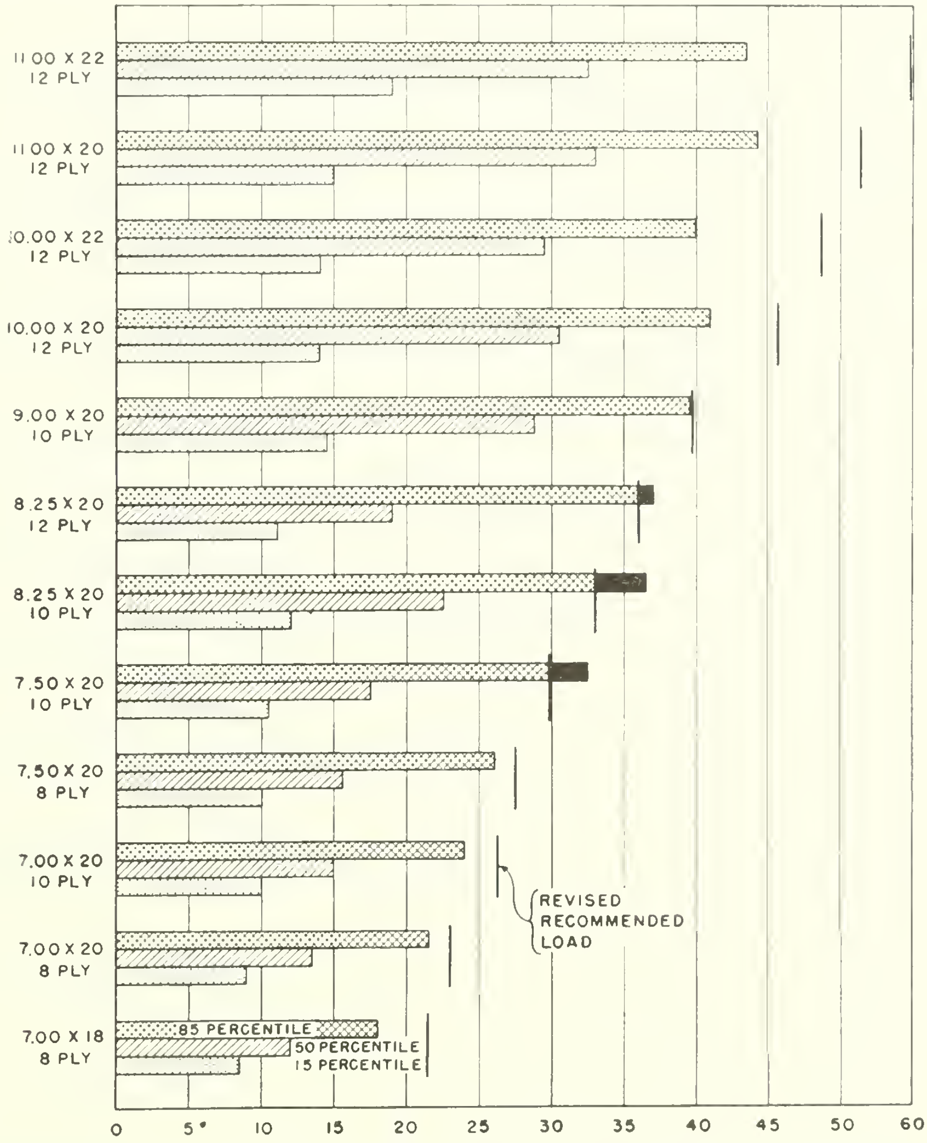

Figure 3.—Comparison of 15-, 50-, and 85-percentile values of tire loads for dual tires.

Table 9.—Comparison of 15-, 50-, and 85-percentile tire loads of dual tires with maximum recommended load

Tire size	Number of ply	Ratio of observed tire load to maximum recommended load—					
		Before Apr. 15, 1955			After Apr. 15, 1955		
		15 percentile	50 percentile	85 percentile	15 percentile	50 percentile	85 percentile
7.00 x 18	8	0.46	0.65	0.98	0.40	0.56	0.84
7.00 x 20	8	.45	.68	1.08	.39	.58	.93
7.00 x 20	10	45	.67	1.07	.38	.57	.92
7.50 x 20	8	.42	.65	1.10	.37	.57	.95
7.50 x 20	10	.39	.65	1.20	.35	.59	1.09
8.25 x 20	10	.41	.78	1.26	.36	.67	1.10
8.25 x 20	12	.35	.60	1.17	.30	.53	1.03
9.00 x 20	10	.42	.83	1.14	.37	.72	1.00
10.00 x 20	12	.35	.76	1.03	.30	.66	.89
10.00 x 22	12	.33	.69	.94	.29	.60	.82
11.00 x 20	12	.33	.73	.98	.29	.64	.86
11.00 x 22	12	.40	.68	.92	.35	.59	.79

loads that are generally well within the recommended ones. This was in decided contrast to what was indicated by the ratios in table 6. In that instance, hot tire pressures were shown to be considerably above the recommended cold inflation pressures.

Relation of Tire Pressure to Tire Load

The results shown in table 10 are based on the analysis of the tire pressures in the class interval of tire load that most nearly corresponds to the new recommended tire load. Included under the tire-load heading are the maximum recommended load and its percentile value in the total sample, the class interval into which it fell, and the number of tire loads in that class interval. The tire pressure information consists of the maximum recommended pressure (new), the average, 15-, and 85-percentile values of the tire pressures in the respective tire-load class interval, and the ratio of the average observed pressure to the maximum recommended pressure.

In table 10, the high percentile values of the recommended tire load amplify the previous discussion concerning the frequency of tire loads. The size of the sample except for the 8.25 x 20, 10-ply, 9.00 x 20, 10-ply, and the 10.00 x 20, 12-ply tires certainly tends to detract from the validity of the results. However, the average and the percentile tire pressures in table 10 appear to be in line with those shown previously for the sample as a whole. A rather wide dispersion of the pressures within a class interval is indicated by the difference between the 15- and 85-percentile values.

One point of interest is the close agreement, where there was a sizable sample, between the values in table 10 and those shown for the total sample in table 4 and figure 2. For example, in the case of the 10.00 x 20, 12-ply dual tires, the average value from table 4 was 81 p. s. i. and the 15- and 85-percentile values from figure 2 were 72 and 94 p. s. i., respectively. Comparable values from table 10 were 83, 71, and 92 p.s.i., respectively. The inference is that tire pressure did not increase materially with tire load. This point is further illustrated by the comparison in figure 4 of the average pressure ratios listed in tables 5 and 10. It is indicated that the ratio for the maximum load interval usually is only slightly greater than the one for total sample.

Table 11 contains results similar to those in table 10 for conditions of tire load and pressure recommended prior to April 15, 1955. The main reason for including this table is to present another set of values for a specific tire-load group that are based on a larger sample. From table 10, the average, 15-, and 85-percentile values are 84, 75, and 90 p.s.i., respectively, for 23 11.00 x 20, 12-ply tires. The respective values from table 11 are 84, 71, and 92 p.s.i. for 181 tires of the same size. The tire-load class interval in the first instance is 5,000 to 5,249 pounds, and it is 4,250 to 4,749 pounds in the latter instance. The results in table 11 tend to substantiate those presented in table 10.

A further evaluation of the variation of tire pressure with tire load for dual tires is made in table 12 and figure 5. The average pressure and number of observations by 500-pound class intervals of tire load are given in table 12 for 6 of the more common truck tires. The increase in pressure is very gradual over the load range and generally amounts to slightly over 10 percent from the lowest to the highest tire load. The rate of pressure increase with

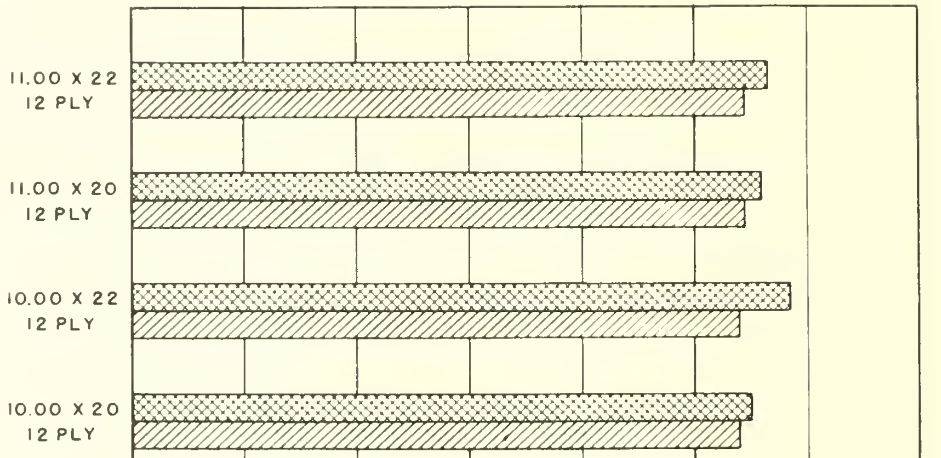

Figure 4.—Comparison of tire pressure ratios for total vehicle sample and for class interval of tire load containing maximum load recommended by the Tire and Rim Association, Inc., after April 15, 1955.

Table 10.—Average, 15-, and 85-percentile values of tire pressure of dual-tired wheels for class interval of tire load corresponding to maximum recommended load after Apr. 15, 1955

Tire size	Number of ply	Tire load: Maximum recommended load: Pounds	Tire load: Maximum recommended load: Percentile	Tire load: Class interval: Pounds	Tire load: Class interval: Number of tires	Tire pressure: Maximum recommended pressure	Tire pressure: Average observed pressure	Tire pressure: 15 percentile	Tire pressure: 85 percentile	Ratio of observed to maximum recommended pressure
						P. s. i.	*P. s. i.*	*P. s. i.*	*P. s. i.*	
7.00 x 18	8	2, 140	95	2, 000–2, 249	6	60	53	[1] 40	[2] 60	0. 88
7.00 x 20	8	2, 310	90	2, 250–2. 499	2	60	68	[1] 65	[2] 70	1. 13
7.00 x 20	10	2, 630	92	2, 500–2. 749	2	75	74	[1] 72	[2] 75	. 99
7.50 x 20	8	2, 740	88	2, 500–2, 749	20	65	70	60	75	1. 07
7.50 x 20	10	2, 980	77	2, 750–2, 999	24	75	71	60	85	. 95
8.25 x 20	10	3, 330	75	3, 250–3, 499	307	70	74	60	83	1. 05
8.25 x 20	12	3, 600	83	3, 500–3, 749	16	80	75	61	86	. 94
9.00 x 20	10	3, 960	85	3, 750–3, 999	198	70	81	66	89	1. 15
10.00 x 20	12	4, 580	95	4, 500–4, 749	466	75	83	71	92	1. 10
10.00 x 22	12	4, 880	99	4, 750–4, 999	39	75	88	75	100	1. 17
11.00 x 20	12	5, 150	96	5. 000–5, 249	23	75	84	75	90	1. 12
11.00 x 22	12	5, 480	97	5, 250–5, 499	18	75	85	80	90	1. 13

[1] Minimum value. [2] Maximum value.

Table 11.—Average, 15-, and 85-percentile values of tire pressure of dual-tired wheels for class interval of tire load corresponding to maximum recommended load before Apr. 15, 1955

Tire size	Number of ply	Tire load: Maximum recommended load: Pounds	Tire load: Maximum recommended load: Percentile	Tire load: Class interval: Pounds	Tire load: Class interval: Number of tires	Tire pressure: Maximum recommended pressure	Tire pressure: Average observed pressure	Tire pressure: 15 percentile	Tire pressure: 85 percentile	Ratio of observed to maximum recommended pressure
						P. s. i.	*P. s. i.*	*P. s. i.*	*P. s. i.*	
7.00 x 18	8	1, 850	87	1. 750–1, 999	10	55	60	40	68	1.09
7.00 x 20	8	2, 000	79	1, 750–2. 249	21	55	62	45	73	1. 12
7.00 x 20	10	2, 250	75	2, 000–2, 499	11	70	64	53	72	. 91
7.50 x 20	8	2, 375	81	2, 250–2, 499	17	60	64	60	70	1. 07
7.50 x 20	10	2, 700	72	2, 500–2, 749	23	70	66	58	80	. 94
8.25 x 20	10	2, 900	63	2, 750–2. 999	167	65	72	59	82	1. 11
8.25 x 20	12	3, 150	70	3, 000–3. 249	8	75	68	60	75	. 91
9.00 x 20	10	3, 450	71	3, 250–3, 499	221	65	77	62	86	1. 18
10.00 x 20	12	4, 000	82	3, 750–4, 249	2, 274	70	84	71	90	1. 20
10.00 x 22	12	4, 275	91	4. 000–4, 499	440	70	85	73	92	1. 21
11.00 x 20	12	4, 500	88	4. 250–4, 749	181	70	84	71	92	1. 20
11.00 x 22	12	4. 750	93	4, 500–4. 999	117	70	86	76	92	1. 23

load is shown in figure 5 for the smallest and largest tire sizes included in table 12.

It seems rather conclusive that tire load is not too significant in the study of tire pressures measured at random from the general traffic stream. Similar investigations for a particular vehicle type and State or region have yielded approximately the same trends. The tendency for a particular tire size is for the pressure to increase with tire load; however, in the range of loads that would normally be considered in highway design, the increase is rather insignificant.

Effect of Vehicle Type and Geographical Area on Tire Pressure

The average tire pressures for the more predominant vehicle types are summarized in table 13 by tire size. For a given tire size the variation of pressure with vehicle type is not material if the less common vehicle combinations are excepted. This is clearly indicated by figure 6, which compares the average pressures for two tire sizes. Excluding truck-tractor-semitrailer and full-trailer combinations (2–S1–2 and 2–S2–2), the range in average pressure was from 74 to 81 p.s.i. for the 10.00 x 20, 12-ply tires.

As indicated earlier in the report, an analysis was made to determine whether there was any regional variation of inflation pressures. This was accomplished by grouping the data by the four AASHO regions. Sample results of the analyses for 10.00 x 20, 12-ply tires, irrespective of vehicle type, are shown in table 14. Also included in the same table are the average inflation pressures by region for the 2-axle truck-tractors with 2-axle semitrailers. On the basis of these results, it may be assumed that the results for the country as a whole may be applied safely to any given region.

Other Tire-Pressure Factors Studied

Ambient temperature

A trial analysis was made on the samples of tire pressures for the 10.00 x 20, 12-ply tires checked in Missouri and Iowa to investigate the relation between ambient temperature and tire pressure. The sample for Missouri was the largest of any State and was observed at 15 widely separated stations. Ambient temperatures for day and night operations ranged from 62° to 108° F. The Iowa sample was one of the largest and all of the tires were checked at the same station. The range in temperatures in this instance was a little less, 62° to 98° F. The results are summarized in table 15 for the more common class intervals of tire loads. As was expected, considering the uncontrollable factors involved, there was no definite trend. In Missouri the pressure tended to increase rather sharply after about

Table 12.—Variation of tire pressure with tire load for dual tires

Tire-load interval	8.25 x 20, 10-ply tires: Number checked	8.25 x 20, 10-ply tires: Pressure	9.00 x 20, 10-ply tires: Number checked	9.00 x 20, 10-ply tires: Pressure	10.00 x 20, 12-ply tires: Number checked	10.00 x 20, 12-ply tires: Pressure	10.00 x 22, 12-ply tires: Number checked	10.00 x 22, 12-ply tires: Pressure	11.00 x 20, 12-ply tires: Number checked	11.00 x 20, 12-ply tires: Pressure	11.00 x 22, 12-ply tires: Number checked	11.00 x 22, 12-ply tires: Pressure
Pounds		*P. s. i.*		*P. s. i.*		*P. s. i.*		*P. s. i.*		*P. s. i.*		*P. s. i.*
1,000–1,499	626	69	425	73	2, 043	77	635	76	183	78	143	77
1,500–1,999	697	69	495	72	1, 603	77	473	78	166	79	164	78
2,000–2,499	412	69	349	73	1, 258	80	420	79	121	80	139	81
2,500–2,999	321	72	364	76	1, 516	80	572	79	143	80	234	81
3,000–3,499	563	75	488	76	2. 110	82	709	83	159	84	313	83
3,500–3,999	388	77	408	80	2, 679	83	709	85	235	84	327	82
4,000–4,499	222	81	291	81	1, 752	85	440	85	230	84	189	83
4,500–4,999	61	77	109	82	642	84	137	85	110	86	117	86
5,000–5,499	6	63	21	81	116	85	29	86	47	84	33	86
5,500–5,999	2	90	2	70	37	84	10	86	17	86	23	88

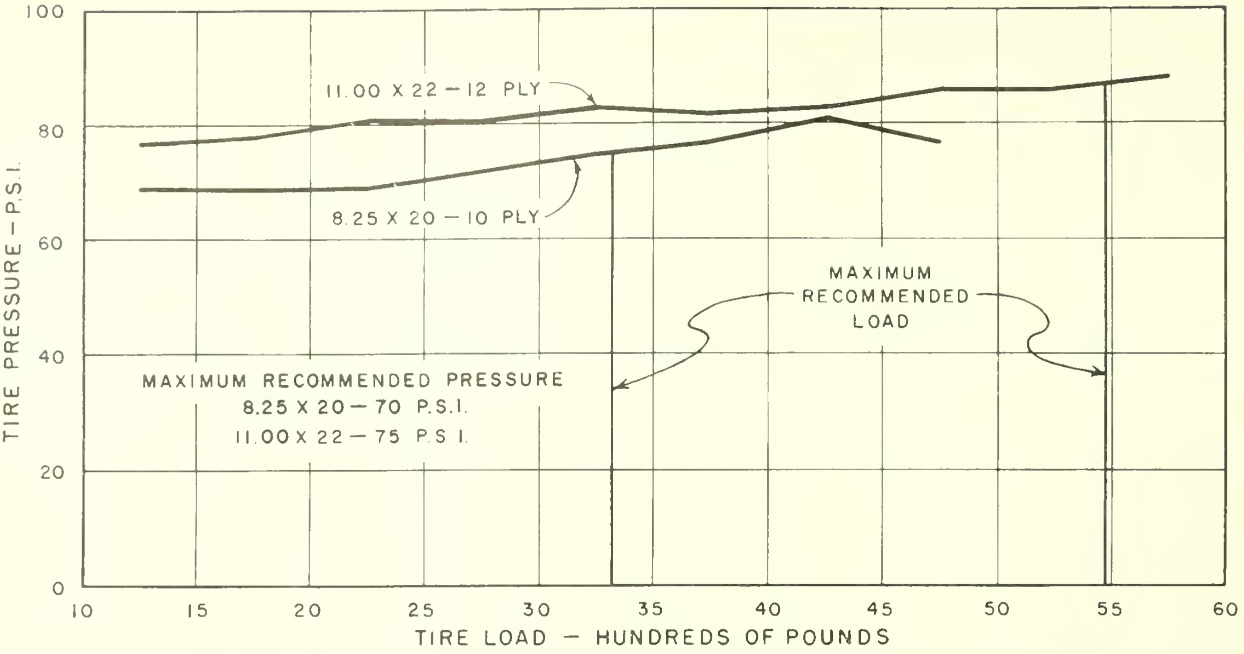

Figure 5.—Variation of tire pressure with load for dual tires of two common sizes.

95° F. It appeared from other investigations of the data that pressures during the daytime period of higher temperatures would seldom be more than 5 p.s.i. above the average of the combined night and day operating pressures.

Motor-carrier tire inflation practices

One of the items recorded on a recent investigation of truck-operation practices was the initial (cold) inflation pressure specified by a motor carrier for his fleet. The frequency of specified pressures and the average values are shown in table 16 for carriers interviewed during 1955 and 1956 in 32 States. Also included for comparison are the recommended pressures, and the 50- and 85-percentile values of pressure read from figure 2 for the given tire size. It is very evident that there is a sizable variation in carrier practice. Tire pressures reported by the average carrier are about 5 p.s.i. above the recommended pressure.

The 50- and 85-percentile values obtained by the tire-pressure survey and reported in table 16 are about 5 and 15 p.s.i., respectively, above the values for the average carrier. In this respect, one tire manufacturer has reported that normal inflation increases would probably be from a recommended value of 75 to 90 or 95 p.s.i. Other available references seem to support such a pressure differential. It would appear that the results of the tire survey are within the expected limits.

Use of high-pressure tires

In the past few years there has been a trend toward the use of higher pressure tires. This has been characterized by the introduction of a "high-load" tire with all-nylon cord in this country and the importing of steel cord tires from Europe. Neither of these types were prominent in the sample collected in 1954. Nevertheless, the trend has been viewed with alarm by some highway engineers.

Since 1954 the use of the high-load tire undoubtedly has increased substantially. It is used by operators who want maximum vertical cargo space in their trailer units. The popular size is the 10.3 x 20, 14-ply (nylon cord equivalent) tire with a maximum recommended load of 4,750 pounds and inflation pressure of 95 p.s.i. when mounted on dual wheels. Seven vehicles equipped with tires of this size were checked in the 1954 survey. The average pressure was 90 p.s.i. which was 9 p.s.i. above the average found for the much larger sample of the comparable conventional tire size. The range in pressure was from 70 to 110 p.s.i.

There have been persistent rumors that some carriers are operating high-load tires at pressures less than the recommended because of alleged operating advantages. One of the largest west coast operators specifies an initial inflation pressure of 85 p.s.i. on the single tires (front axle) and 80 on the dual tires. Another fleet in Illinois inflates to 90 p.s.i. From the limited data, it would appear that pressures of at least 10 p.s.i. in excess of those found for the comparable conventional size may be expected, but probably not to the extent of the difference in recommended inflation pressures which is 20 p.s.i.

The steel cord tire has 2 to 4 plies which replace the usual 10 to 18 of the textile type. It is believed to be in very limited use in this country at the present time. In the 1954

Table 13.—Average observed air pressure for dual tires by tire size and vehicle type

Tire size	Number of ply	Average observed air pressure of dual tires, by vehicle types										All vehicles	
		2D	3	2-S1	2-S2	3-S2	2-S1-2	2-S2-2	2-2	3-2	3-3	Number of tires	Average air pressure
		P. s. i.	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*		*P. s. i.*
7.00 x 18	8	51	------	------	------	------	------	------	------	------	------	112	51
7.00 x 20	8	57	------	------	------	------	------	------	------	------	------	104	57
7.00 x 20	10	62	------	------	------	------	------	------	------	------	------	61	62
7.50 x 20	8	62	------	68	------	------	------	------	------	------	------	415	62
7.50 x 20	10	64	------	78	------	------	------	------	------	------	------	528	67
8.25 x 20	10	69	75	74	78	------	------	------	------	------	------	3, 556	72
8.25 x 20	12	71	78	74	------	------	------	------	------	------		244	73
9.00 x 20	10	73	78	75	79	75	------	------	------	------	------	3, 054	76
10.00 x 20	12	77	74	80	81	79	89	89	------	81	74	14, 250	81
10.00 x 22	12	------	------	83	83	78	83	94	86	80	75	4, 256	81
11.00 x 20	12	76	------	80	83	82	87	94	------	82	------	1, 451	82
11.00 x 22	12	78	------	83	82	79	86	------	------	78	------	1, 739	82
11.00 x 24	12	90	------	80	86	------	------	------	------	------	------	36	84

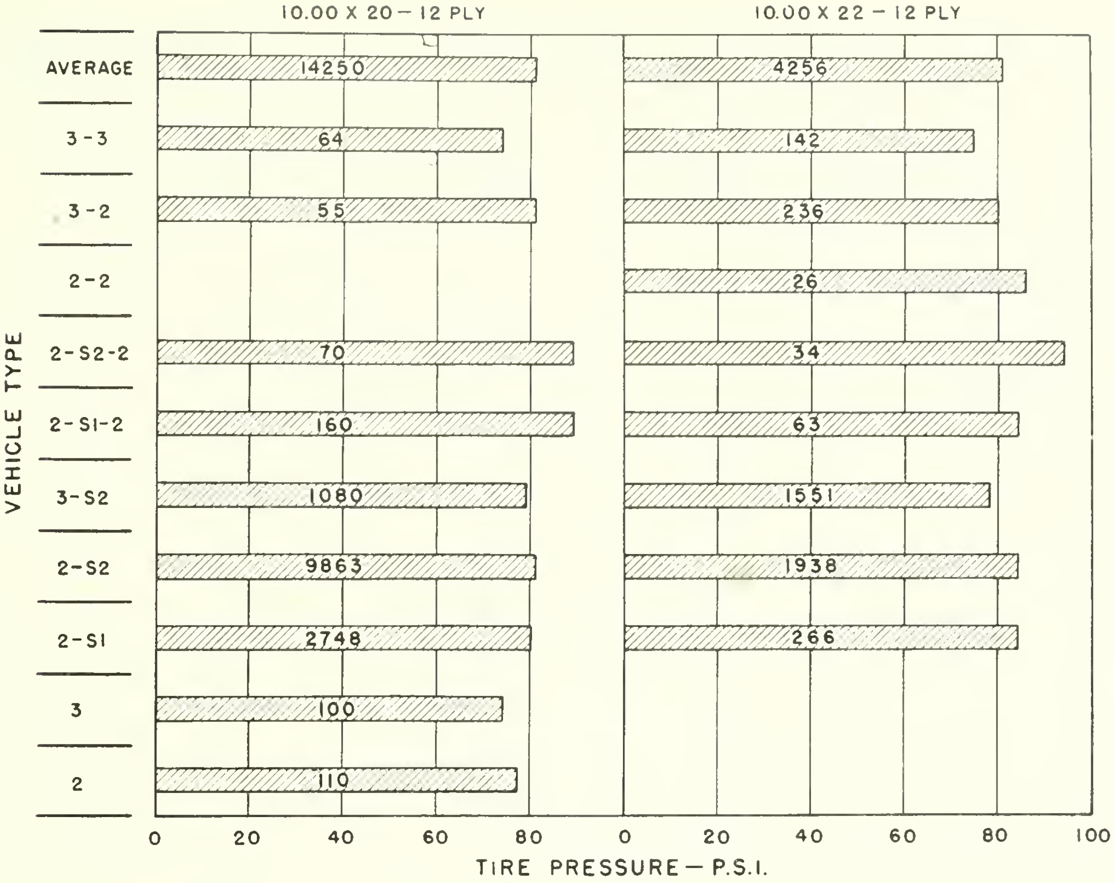

Figure 6* (above).—*Variation of tire pressure with vehicle type for dual tires of two sizes.

survey one combination was found with this type of tire. The average pressure for the tires checked was 116 p.s.i. The size encountered has a maximum load rating of 6,500 pounds per tire when inflated to 110 p.s.i. and used as duals. The rating is 5,400 pounds if inflated to 100 p.s.i. which compares closely with a rating of 5,480 pounds for a conventional 11.22 x 20, 12-ply tire inflated to 75 p.s.i. The steel cord tire has the same operating advantage over the conventional tire as the high-load tire. That is, it is possible to carry the same wheel load on a smaller diameter tire.

Table 14.—Variation of tire pressure for 10.00 x 20, 12-ply dual tires, by regions

Tire-load interval (pounds)	Average tire pressure for all vehicle types observed in—					Average tire pressure for 2-axle truck-tractors with 2-axle semitrailers (2-S2) observed in—				
	Region 1	Region 2	Region 3	Region 4	All regions	Region 1	Region 2	Region 3	Region 4	All regions
	P. s. i.	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*
1,000–1,499	78	78	77	74	77	77	78	78	76	78
1,500–1,999	78	76	77	76	77	79	78	78	78	78
2,000–2,499	79	81	79	78	80	81	84	79	77	81
2,500–2,999	80	80	80	77	80	81	78	80	78	80
3,000–3,499	83	83	81	80	82	84	83	82	82	82
3,500–3,999	83	83	83	81	83	84	83	87	84	85
4,000–4,499	84	83	87	82	85	85	83	87	84	85
4,500–4,999	83	82	86	84	84	87	81	84	83	83
5,000–5,499	87	83	86	84	85	90	83	90	82	85
5,500–5,999	84	----	----	85	84	79	----	----	85	80
Average air pressure	81	81	81	79	81	82	81	81	80	81

Table 15.—Variation of tire pressure with ambient temperature for 10.00 x 20, 12-ply dual tires

Tire-load interval (pounds)	Number of tires	Average tire pressure for temperatures (° F.) ranging from—									Average pressure
		60–64 degrees	65–69 degrees	70–74 degrees	75–79 degrees	80–84 degrees	85–89 degrees	90–94 degrees	95–100 degrees	100–104 degrees	
MISSOURI											
		P. s. i.	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*
2,500–2,999	213	------	75	82	81	80	83	80	81	------	81
3,000–3,499	263	81	75	81	80	80	87	83	88	91	82
3,500–3,999	464	80	81	77	78	85	84	83	84	87	82
4,000–4,499	242	82	82	82	83	84	84	80	89	87	84
Average air pressure	1, 182	81	79	80	80	83	84	82	85	88	82
IOWA											
2,500–2,999	171	79	79	76	79	------	------	86	77	------	78
3,000–3,499	195	78	80	79	80	------	80	83	85	------	81
3,500–3,999	241	81	81	80	82	------	87	90	78	------	82
4,000–4,499	134	81	89	79	82	------	81	80	80	------	81
Average air pressure	741	80	82	79	81	------	83	85	80	------	81

Table 16.—Initial (cold) inflation pressures reported by selected carriers compared with results of the present survey made on the highways

Tire size	Number of carriers	Tire pressure				Percentage of carriers reporting initial (cold) inflation pressure of—							
		Maximum recommended pressure	Average pressure reported by carriers	Observed pressure		60 p. s. i.	65 p. s. i.	70 p. s. i.	75 p. s. i.	80 p. s. i.	85 p. s. i.	90 p. s. i.	95 p. s. i. and over
				50 percentile	85 percentile								
		P. s. i.	*P. s. i.*	*P. s. i.*	*P. s. i.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
9.00	71	70	76	79	90	-----	16.9	19.7	26.8	16.9	11.3	8.4	------
10.00	418	75	79	83	94	1.2	2.6	17.7	21.5	30.4	16.0	8.9	1.7
11.00	85	75	80	85	95	1.2	2.3	14.1	21.2	27.1	15.3	17.6	1.2

Table 17.—Results of 1940 survey in Kansas compared with the present survey

Tire size	Number of ply	Tire pressure					
		Maximum recommended pressure		Observed pressure, 50 percentile		Observed pressure, 85 percentile	
		1940	1954 [1]	1940	1954	1940	1954
		P. s. i.	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*	*P. s. i.*
7.00 x 20	8	55	55	52	60	63	72
7.50 x 20	8	55	60	57	65	72	77
8.25 x 20	10	60	65	72	74	89	87
9.00 x 20	10	65	65	76	79	90	89
9.75 x 20	12	70	[2] 70	81	[2] 83	94	[2] 94

[1] Recommended air pressure prior to Apr. 15, 1955.
[2] 10.00 x 20, 12-ply tires.

Results of 1940 and 1954 Surveys Compared

The results of a tire-pressure survey published in 1941 by the Kansas State Highway Commission are compared with the results of the present survey in table 17. The 50- and 85-percentile values for the 1954 survey are read from figure 2. There is little difference in the values for the three larger tire sizes; however, the 1954 values are appreciably higher in the case of the 7.00 and 7.50 sizes.

The significant variation between the results of the two surveys was in the percentage of the tire loads for a given tire size that exceeded the maximum recommended load. For example, in the earlier study 41 percent of the tire loads for 9.00 x 20, 10-ply tires were in excess of the recommended value. In 1954, the comparable percentage was 29 percent for the old schedule and 15 percent for the new schedule of recommended tire loads.

State Highway-User Taxes Paid in 1954 and 1955 on Vehicles of Various Type and Weight Groups

BY THE DIVISION OF FINANCIAL AND ADMINISTRATIVE RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by ELIZABETH SAMSON,
Transportation Economist

This study of highway-user taxes contributed by vehicles of different type and weight groups is primarily concerned with revenues received by the States in 1954, although preliminary data for 1955 are also included.

State highway-user taxes collected by the States during 1954 amounted to $3,623 million, of which 63.6 percent came from motor-fuel taxes, 29.4 percent from registration fees, 2.3 percent from motor-carrier taxes, 1.9 percent from operator's and chauffeur's licenses, and 2.8 percent from miscellaneous fees and taxes. Revenues from the same sources amounted to $4,008 million in 1955, a 10.6-percent increase over the previous year.

In 1954, passenger cars contributed 64.7 percent of the total State highway-user taxes; trucks and truck combinations, 32.9 percent; buses, 1.8 percent; and motorcycles and light trailers, 0.6 percent.

Excluding motorcycles and light trailers from consideration, passenger cars accounted for 83.5 percent of the registrations, 81.0 percent of the mileage traveled, and 65.1 percent of the user-tax payments; trucks and truck combinations represented 16.3 percent of the registrations, 18.3 percent of the travel, and 33.1 percent of the tax payments; and buses accounted for 0.2 percent of the registrations, 0.7 percent of the travel, and 1.8 percent of the tax payments.

When 2-axle, 4-tire single-unit trucks are combined with passenger cars and compared with medium and heavy trucks, the percentages of registration, travel, and tax payments were as follows: light vehicles, 93.7, 90.6, and 74.3 percent; and medium and heavy trucks, 6.1, 8.8, 23.9 percent, respectively.

Average payments of highway-user taxes in 1954 were made at the following rates per vehicle, per vehicle-mile, and per ton-mile: passenger cars $48, 0.52 cent, and 0.26 cent; trucks and truck combinations $127, 1.17 cents, and 0.17 cent; and buses $462, 1.83 cents, and 0.17 cent, respectively. For truck combinations alone, the corresponding amounts were $850, 2.20 cents, and 0.12 cent.

In 1955, average user-tax payments per vehicle, per vehicle-mile, and per ton-mile were $50, 0.54 cent, and 0.27 cent for passenger cars; $131, 1.21 cents, and 0.18 cent for trucks and truck combinations; and $470, 1.85 cents, and 0.20 cent for buses. Truck combinations alone contributed $881, 2.25 cents, and 0.12 cent, respectively.

IN 1954 the Bureau of Public Roads published estimates of State highway-user taxes paid during 1952 on vehicles in different type and weight groups.[2] Since better and more up-to-date information is now available, it has been deemed desirable to undertake similar computations for the calendar year 1954. A complete analysis could not be made for 1955, but some preliminary estimates for that year are included.

The estimates given in this article cover the continental United States and include only payments that are generally regarded as State highway-user taxes, as follows: motor-vehicle registration fees; motor-carrier taxes; operator's and chauffeur's licenses; motor-fuel taxes, including imposts levied on special fuels; and miscellaneous fees and charges, such as certificate-of-title fees, special titling taxes, and service charges. State-imposed sales and excise taxes, fines and penalties, and toll charges are omitted, as are all imposts levied directly against motor vehicles and their use by the Federal Government, counties, cities, special authorities, and other local units.

State highway-user taxes totaled $3,623 million for 1954 and $4,008 million for 1955. These amounts exclude $17 million and $18 million, respectively, of fines and penalties, which are not considered to be road-user revenues even though they are miscellaneous receipts of the highway departments in some States. No estimate has been made of income from sales and use taxes on motor vehicles and parts, but the sums received from other imposts on highway users that are not covered by this analysis are given in table 1.

The classification of vehicles which was selected as having the most meaning for this study is essentially the one by which vehicles are recognized on the highways and by which traffic data are usually collected and presented. The principal modification in this visual classification was made to subclassify buses so that the distribution of highway-user taxes would reflect differences in the taxes required for buses in three different types of operation—school bus service and commercial intercity and transit service.

Findings

Of the total of $3,623 million received by the States from highway-user taxes in 1954, 63.6 percent or $2,306 million came from motor-fuel taxes, 29.4 percent or $1,064 million from registration fees, 2.3 percent or

Table 1.—Revenue from highway-user imposts not included in this analysis

Type of impost	Amounts paid in—	
	1954	1955
	Million dollars	*Million dollars*
Federal motor-fuel, lubricating oil, and excise taxes [1]	2, 204	2, 736
State-imposed toll charges	167	211
County and local imposts on motor-vehicle users	54	58
County and local toll charges	58	60
Total	2, 483	3, 065

[1] These amounts are the portions of the Federal imposts estimated to have been paid by highway users. The gross collections were $2,334 million in 1954 and $2,867 million in 1955.

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[1] Estimates of the distribution of vehicles by types and their corresponding travel were prepared by Nathan Lieder, statistician, Bureau of Public Roads.

[2] *Estimate of user taxes paid by vehicles in different type and weight groups*, by Edwin M. Cope, John T. Lynch, and Clarence A. Steele. PUBLIC ROADS, vol. 28, No. 2, June 1954, pp. 17-26. The same article was published in a somewhat revised and expanded form but without change in the basic data in *Highway-User Taxation*, Bulletin 92, Highway Research Board, pp. 15-34.

Table 2.—Estimate of State highway-user taxes paid in 1954, by major types of vehicles

Vehicle type	Registration fees	Motor-carrier taxes	Operator's and chauffeur's license fees	Miscellaneous fees	Motor-fuel taxes	Total	
						Amount	Distribution
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*Percent*
Passenger cars	603, 446	-------	55, 821	81, 487	1, 602, 572	2, 343, 326	64. 7
Motorcycles	1, 840	-------	135	513	5, 027	7, 515	. 2
Camp and other light trailers	15, 633	-------	-------	-------	-------	15, 633	. 4
School and miscellaneous buses	1, 735	-------	247	125	2, 283	4, 390	. 1
Commercial buses:							
Intercity	4, 050	9, 360	100	51	9, 891	23, 452	. 6
Transit	6, 791	(1)	260	132	29, 629	36, 812	1. 1
All commercial buses	10, 841	9, 360	360	183	39, 520	60, 264	1. 7
All buses	12, 576	9, 360	607	308	41, 803	64, 654	1. 8
Single-unit trucks:							
2-axle, 4-tire	110, 267	-------	6, 278	8, 200	207, 034	331, 779	9. 1
2-axle, 6-tire	168, 415	3, 396	3, 105	6, 253	174, 777	355, 946	9. 8
3-axle	33, 597	331	337	1, 230	28, 084	63, 579	1. 8
All single-unit trucks	312, 279	3, 727	9, 720	15, 683	409, 895	751, 304	20. 7
Vehicle combinations:							
Tractor-semitrailer	105, 529	64, 025	1, 080	4, 100	216, 787	391, 521	10. 9
Truck-trailer	12, 922	5, 715	135	410	29, 675	48, 857	1. 3
All combinations	118, 451	69, 740	1, 215	4, 510	246, 462	440, 378	12. 2
All trucks and combinations	430, 730	73, 467	10, 935	20, 193	656, 357	1, 191, 682	32. 9
All vehicles	1, 064, 225	82, 827	67, 498	102, 501	2, 305, 759	3, 622, 810	100. 0

[1] In many States intracity buses are exempt from most, if not all, State carrier taxes, and it was not possible to segregate the amount that would be paid on these vehicles.

$83 million from motor-carrier taxes, and 1.9 percent or $67 million from operator's and chauffeur's license fees. The remaining 2.8 percent or $103 million came from a variety of miscellaneous sources, the most productive of which were certificate-of-title fees and special titling taxes.

In 1955 State highway-user taxes amounted to $4,008 million, 10.6 percent more than they produced the year before. Although contributions to the increase varied from one type of tax to another, no significant change occurred in the percentage distribution of total user taxes among their principal sources. Registration fees, motor-carrier taxes, and motor-fuel taxes, each of which rose in the neighborhood of 10 percent, amounted to $1,162 million, $91 million, and $2,543 million, respectively, in 1955. Operator's and chauffeur's license fees increased 30 percent to $87 million, and miscellaneous fees and taxes amounted to $124 million, an increase of 21 percent.

The findings of this study are presented first for 1954 in tables 2 and 3, portrayed graphically in figures 1–3, and then for 1955 in tables 4 and 5. In the interpretation of these data it should be borne in mind that they are nationwide totals and averages derived by processing in various ways the information reported by the 48 States and the District of Columbia. Each State has its own schedule of user taxes, with the rates of payment differing widely. The vehicles of each type and size group may contribute relatively more in one State and relatively less in another. This study summarizes the situation as a whole, giving approximate values of the aggregate and average payments by each vehicle group and thereby affording comparisons of the extent to which each group shares in the total burden of State road-user taxation.

Table 3.—Estimate of average State highway-user taxes paid in 1954 per vehicle, per vehicle-mile, and per ton-mile

Vehicle type	Motor vehicles registered [1]		Vehicle-miles traveled [1]		Highway-user taxes paid		Average rate of payment		
	Number	Distribution	Amount	Distribution	Amount [2]	Distribution	Per vehicle	Per vehicle-mile	Per ton-mile [3]
	Thousands	*Percent*	*Millions*	*Percent*	*1,000 dollars*	*Percent*	*Dollars*	*Cents*	*Cents*
Passenger cars	48, 324	83. 50	448, 913	81. 03	2, 343, 326	65. 10	48	0. 52	0. 26
School and miscellaneous buses	57	. 10	343	. 06	4, 390	. 12	77	1. 28	. 22
Commercial buses:									
Intercity	23	. 04	1, 206	. 22	23, 452	. 65	1, 020	1. 94	. 17
Transit	60	. 10	1, 990	. 36	36, 812	1. 02	614	1. 85	. 16
All commercial buses	83	. 14	3, 196	. 58	60, 264	1. 67	726	1. 89	. 16
All buses	140	. 24	3, 539	. 64	64, 654	1. 79	462	1. 83	. 17
Single-unit trucks:									
2-axle, 4-tire	5, 882	10. 17	52, 742	9. 52	331, 779	9. 21	56	. 63	. 27
2-axle, 6-tire	2, 824	4. 88	25, 998	4. 69	355, 946	9. 89	126	1. 37	. 23
3-axle	188	. 32	2, 770	. 50	63, 579	1. 77	338	2. 30	. 19
All single-unit trucks	8, 894	15. 37	81, 510	14. 71	751, 304	20. 87	84	. 92	. 24
Vehicle combinations:									
Tractor-semitrailer	471	. 81	17, 929	3. 24	391, 521	10. 88	831	2. 18	. 12
Truck-trailer	47	. 08	2, 100	. 38	48, 857	1. 36	1, 040	2. 33	. 10
All combinations	518	. 89	20, 029	3. 62	440, 378	12. 24	850	2. 20	. 12
All trucks and combinations	9, 412	16. 26	101, 539	18. 33	1, 191, 682	33. 11	127	1. 17	. 17
All vehicles	57, 876	100. 00	553, 991	100. 00	3, 599, 662	100. 00	62	. 65	. 22
Regrouping of vehicle types:									
Passenger cars and light trucks [4]	54, 206	93. 67	501, 655	90. 55	2, 675, 105	74. 32	49	. 53	. 26
Medium and heavy trucks and combinations	3, 530	6. 09	48, 797	8. 81	859, 903	23. 89	244	1. 76	. 15

[1] Private and commercial motor vehicles only. Publicly owned vehicles, motorcycles, and light trailers are omitted, except for publicly owned transit buses.

[2] Public Roads table DF, 1954, gives $3,639,899,000 as the amount of State imposts on highway users collected in 1954. Omitted from the amounts given in this column are fines and penalties amounting to $17,089,000, tax payments of $15,633,000 assigned to light trailers, and $7,515,000 assigned to motorcycles.

[3] Per ton-mile of average operating gross weight (for average operating gross weights, see table 9, p. 287).

[4] Two-axle, 4-tire trucks are grouped with passenger cars.

1954 State User-Tax Payments

Distribution by type of tax

Estimates of the portions of the major types of user taxes contributed by vehicles of various types and sizes in 1954 are presented in table 2. Of the $3,623 million total, passenger cars are estimated to have provided $2,343 million or 64.7 percent, but their contribution varied somewhat from one type of tax to another. They accounted for 56.7 percent of the registration fees, 69.5 percent of the motor-fuel taxes, and 80.8 percent of all other State highway-user taxes except motor-carrier taxes.

Trucks and truck combinations provided an estimated $1,192 million or 32.9 percent of all user taxes. They accounted for 40.5 percent of the registration fees and 28.5 percent of the motor-fuel taxes. By the very nature of the motor-carrier tax, the portion assigned to trucks and truck combinations would be the greatest part of the total, an estimated 88.7 percent in 1954, although the actual amount is not large when compared with the truck contribution to the other major types of user taxes.

Buses, being relatively few, made a comparatively small contribution to total highway-user taxes, $65 million or 1.8 percent in 1954. They accounted for 1.8 percent of the motor-fuel taxes, a somewhat smaller portion of registration fees, and a negligible portion of miscellaneous user taxes. The fact that buses accounted for as much as 11.3 percent of the motor-carrier taxes is the result of the more restricted tax base of that particular form of taxation.

The remaining State highway-user taxes were contributed by motorcycles and light trailers—$7.5 million and $15.6 million, respectively. These amounts together comprise less than 1 percent of all State highway-user taxes.

Registrations, travel, and taxes paid

Table 3 and figures 1–3 show the highway-user taxes paid on vehicles of various types and sizes in relation to the numbers of these vehicles registered and the distances they traveled. The total tax figure used here is that given in table 2 less the $23 million assigned to motorcycles and light trailers.

Passenger cars accounted for 83.5 percent of all vehicles registered in 1954, 81.0 percent

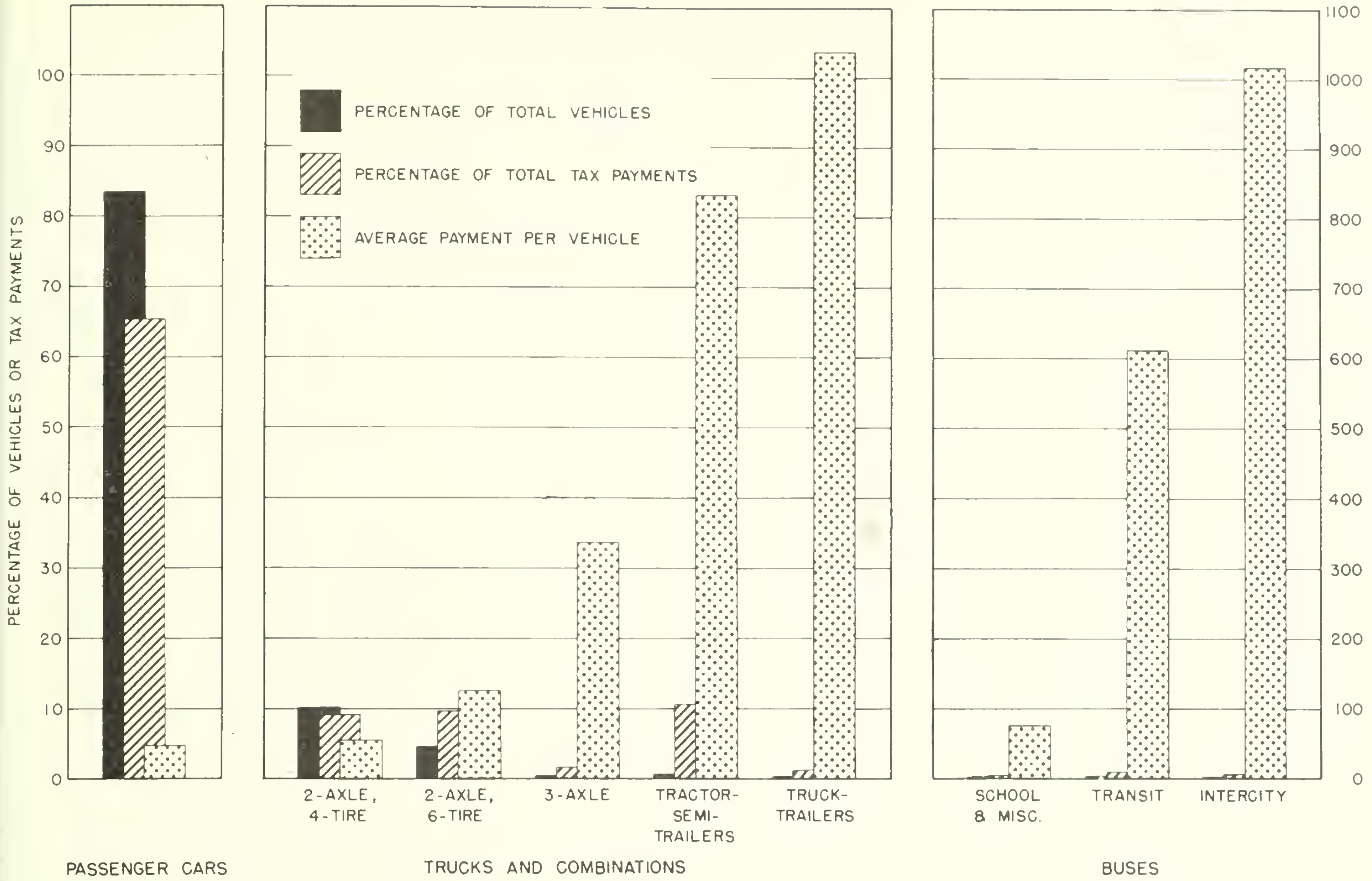

Figure 1.—Comparison of estimated registrations, tax payments, and payments per vehicle in 1954, by vehicle types.

of the mileage traveled, and 65.1 percent of the highway-user taxes. Trucks and truck combinations represented 16.3 percent of the vehicles registered, 18.3 percent of the travel, and 33.1 percent of the user taxes. Buses, which are negligible in the gross totals both as to the number of vehicles and the extent of travel, provided 1.8 percent of the user taxes, though their tax payments per vehicle were relatively high.

The somewhat different grouping of vehicles given at the bottom of table 3 brings out more clearly the relation between numbers of vehicles and tax payments. Light vehicles, comprising automobiles and 2-axle, 4-tire single-unit trucks, constituted 93.7 percent of the registered vehicles in 1954, provided 90.6 percent of the travel, and accounted for 74.3 percent of the user taxes. Medium and heavy trucks and truck combinations accounted for 6.1 percent of the vehicles, 8.8 percent of the travel, and 23.9 percent of the taxes. This grouping of light trucks with passenger cars reduces the truck contribution in absolute amount. On the other hand, the percentages of tax payments and of numbers of vehicles are in a 4 to 1 ratio compared with a 2 to 1 ratio when light trucks are included in the truck category.

Some of the values for classes of vehicles within the major types in the visual classification are significant. Two-axle, 6-tire trucks represented 4.9 percent of the total number of vehicles, but contributed 9.9 percent of the taxes. Three-axle trucks, while accounting for 0.3 percent of the vehicles, made 1.8 percent of the tax payments. Truck combinations as a group comprised only 0.9 percent of the vehicles, but accounted for 12.2 percent of the tax payments. Most of this was contributed by tractor-semitrailers, the use of truck-trailers being restricted through the application of State size and weight laws.

Average rates of payment

Table 3 also compares average rates of payment of highway-user taxes per vehicle, per vehicle-mile, and per ton-mile. These averages are represented by the dot-stippled bars in figures 1, 2, and 3, respectively.

Taxes on passenger cars averaged $48 per vehicle, compared with $127 for trucks and truck combinations and $462 for buses. Within the class comprising trucks and truck combinations are found values of $56 per vehicle for 2-axle, 4-tire trucks; $126 for 2-axle, 6-tire trucks; and about $338 for 3-axle trucks. The average payment for truck combinations as a group was $850, that for tractor-semitrailers alone being $831 and for truck-trailers, $1,040. The high value for truck-trailers arises from the fact that combinations involving full trailers are generally larger and heavier than tractor-semitrailer combinations. Also, the area in which truck-trailers are permitted to operate is limited, and their use tends to be concentrated in States where the tax rates are comparatively high.

When the averages are expressed in terms of payments per vehicle-mile, passenger cars show a value of 0.5 cent; trucks and truck combinations as a group, 1.17 cents; and buses, 1.83 cents. For passenger cars and light trucks combined, the average payment was 0.5 cent per vehicle-mile, while for medium and heavy trucks and truck combinations it was 1.76 cents.

The average payment per vehicle-mile for 2-axle, 4-tire trucks was 0.63 cent, a little more than that for passenger cars. Payments for 2-axle, 6-tire trucks were 1.37 cents and 3-axle trucks, 2.30 cents. The average payment for all single-unit trucks was 0.92 cent. The rate of 2.20 cents per vehicle-mile for combinations as a group was close to the averages for each of the component groups, 2.18 cents for tractor-semitrailers and 2.33 cents for truck-trailer combinations.

Average rates of payment per gross ton-mile, based on average operating gross weight, are also given in table 3. On this basis passenger-car payments averaged 0.26 cent, compared with 0.17 cent for buses and the same amount for trucks and truck combinations. Payments for passenger cars and light trucks together were the same per ton-mile as passenger cars alone, and payments for medium and heavy trucks as a group were 0.15 cent, which is less than the average for all trucks and combinations.

As the average of the two groups indicates, payments for 2-axle, 4-tire trucks were about

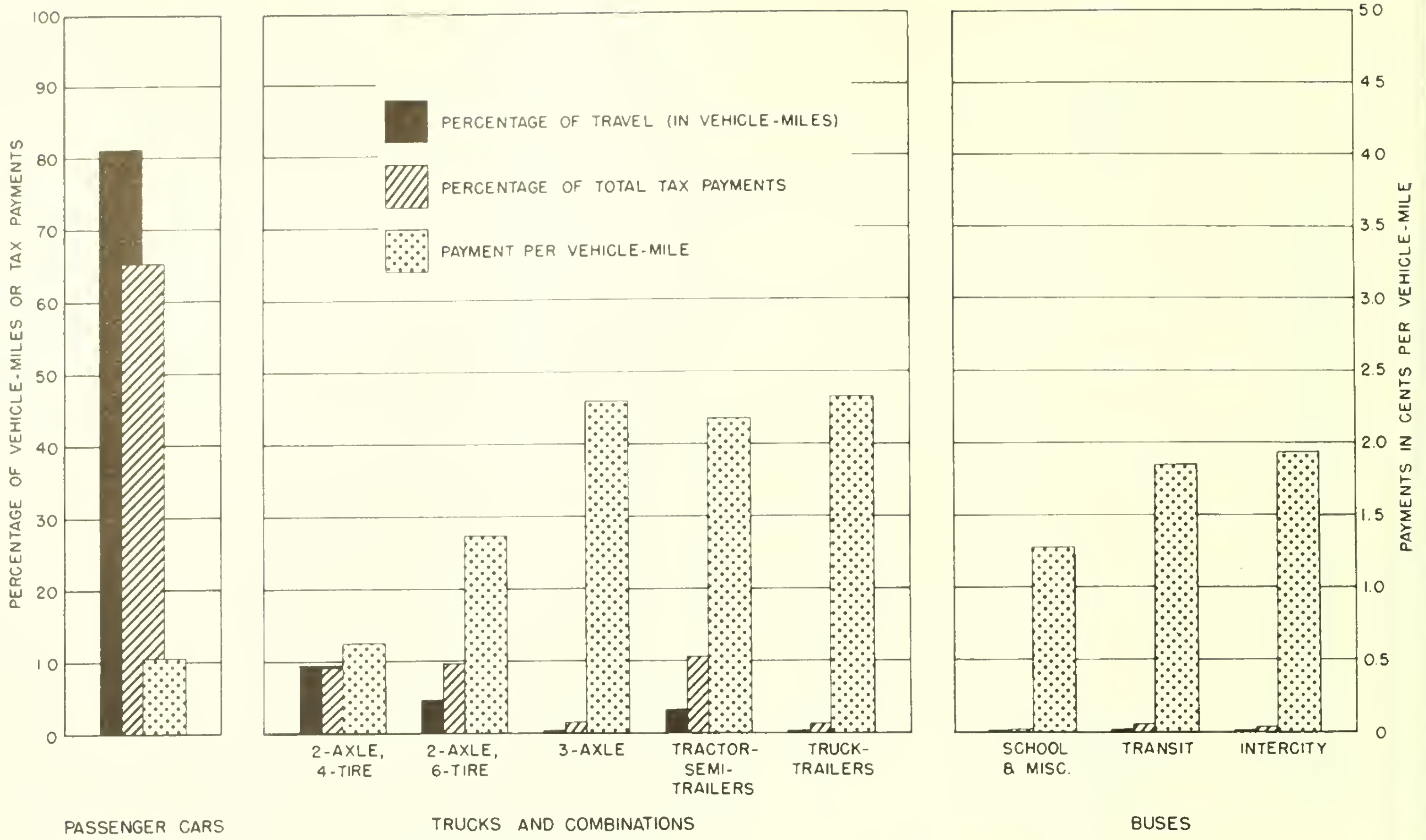

Figure 2.—Comparison of estimated travel, tax payments, and payments per vehicle-mile in 1954, by vehicle types.

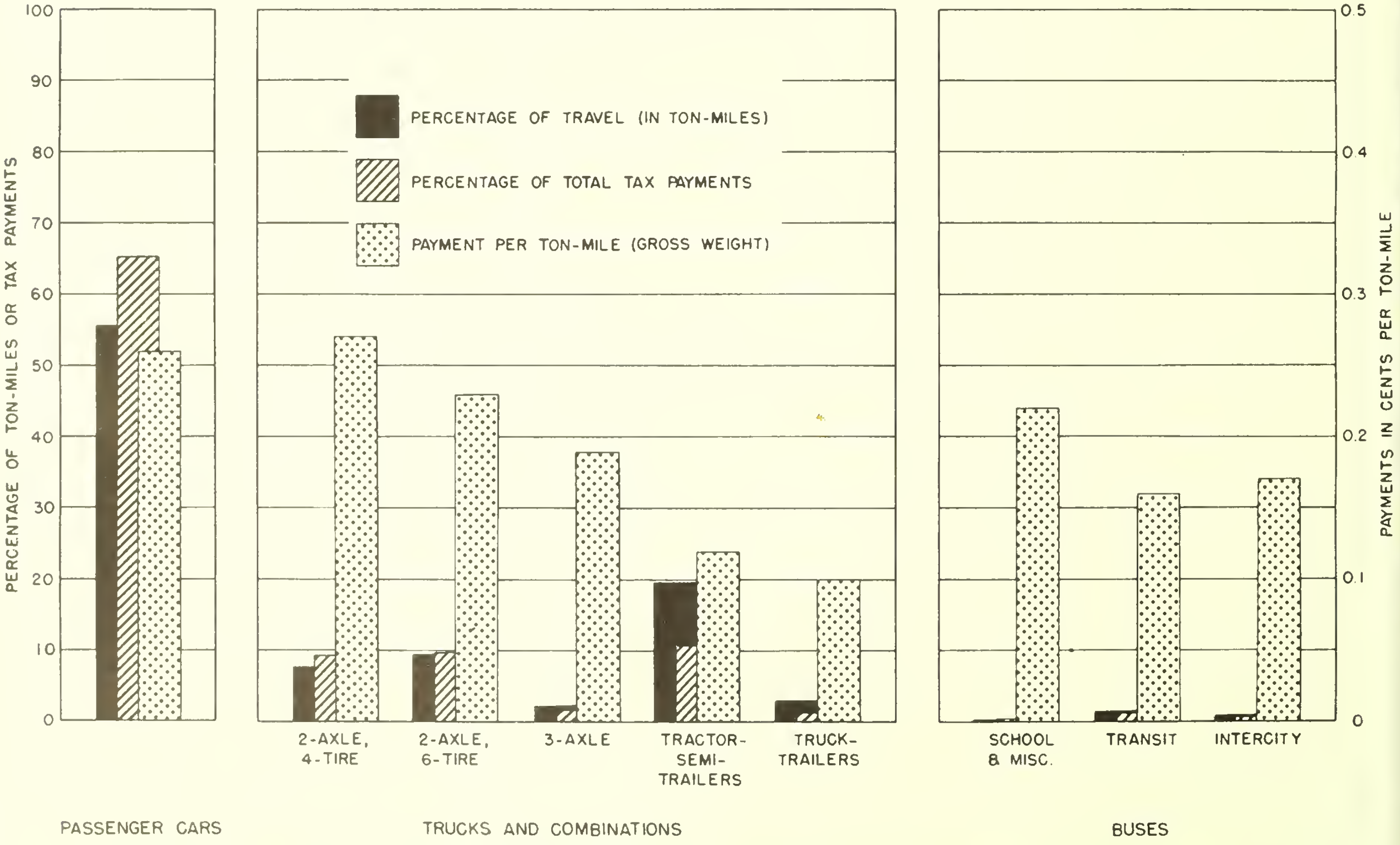

Figure 3.—Comparison of estimated travel, tax payments, and payments per ton-mile in 1954, by vehicle types.

Table 4.—Estimate of State highway-user taxes paid in 1955, by major types of vehicles

Vehicle type	Registration fees	Motor-carrier taxes	Operator's and chauffeur's license fees	Miscellaneous fees	Motor-fuel taxes	Total	
						Amount	Distribution
	1,000 dollars	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*1,000 dollars*	*Percent*
Passenger cars	665, 844	------	72, 296	98, 590	1, 784, 938	2, 621, 668	65. 4
Motorcycles	1, 949	------	175	620	5, 264	8, 008	. 2
Camp and other light trailers	16, 897	------	------	-------	---------	16, 897	. 4
Buses	11, 725	10, 288	787	372	43, 537	66, 709	1. 7
Single-unit trucks	337, 750	4, 097	12, 588	18, 974	441, 676	815, 085	20. 3
Combinations	128, 112	76, 659	1, 573	5, 456	267, 630	479, 430	12. 0
All trucks and combinations	465, 862	80, 756	14, 161	24, 430	709, 306	1, 294, 515	32. 3
All vehicles	1, 162, 277	91, 044	87, 419	124, 012	2, 543, 045	4, 007, 797	100. 0

the same per ton-mile as passenger cars. For 2-axle, 6-tire trucks the average payment was 0.23 cent per ton-mile, and for 3-axle trucks it was 0.19 cent. The difference was slight between the two types of truck combinations, 0.12 cent per ton-mile for tractor-semitrailers and 0.10 cent for truck-trailer combinations.

Comparison between 1952 and 1954]

Although increased State revenue from highway-user taxes between 1952 and 1954 reflected increases in the number of vehicles registered and in the extent of their travel, rising tax rates also had a significant effect. User-tax payments on passenger cars rose from an average of $46 per vehicle in 1952 to $48 in 1954, and from 0.49 to 0.52 cent per vehicle-mile. Buses, which showed little change in numbers or in travel during the period, accounted for tax payments of $404 per vehicle in 1952 compared with $462 in 1954, and payments per vehicle-mile rose from 1.64 to 1.83 cents. For trucks and truck combinations, the increase was from $116 to $127 per vehicle and from 1.10 to 1.17 cents per vehicle-mile.

New data available since 1952 were not sufficient to justify redistributing the numbers of trucks and combinations by visual class. Consequently, the percentages developed in the 1952 analysis were applied to the 1954 registration totals to obtain the distribution for the later year. In the case of buses, new information on vehicle distribution and operating characteristics obtained from industry sources made it possible to subclassify the 1954 data.

Like the bus data, the estimates of travel developed for this analysis are believed to represent considerable improvement over those reported in the earlier study. The article "Traffic and Travel Trends, 1954,"[3] provided an estimate of total motor-vehicle travel in the United States during 1954 and a further breakdown, by visual classes, of the travel of trucks and truck combinations on main rural roads only. In order to obtain a similar classification of travel by trucks and truck combinations on secondary and local rural roads and city streets, percentages developed from the findings of motor-vehicle-use studies recently made in nine States were applied to the total travel by all types of vehicles.

Except for commercial buses, the rates of fuel consumption applied in this study were determined in the same manner as those used in the 1952 analysis. However, more recent information available from State weighing operations warranted minor modifications in the operating weights used for trucks and combinations as described subsequently. Data on average operating weights and fuel consumption supplied by the bus industry were used in the analyses made for commercial buses.

Preliminary Estimates for 1955

Insufficient information is available to allocate 1955 user taxes among classes of vehicles in the complete range used for 1954, but preliminary estimates made for the major types of vehicles—passenger cars, trucks, and buses—are indicative of the 1955 pattern. The available evidence points to the fact that although the level of tax revenues was higher in 1955 than in 1954, the percentage relations have changed little, whether these revenues are distributed by type of tax or by class of vehicle on which the tax payments are made. Table 4, which corresponds to table 2, shows the portions of the principal types of user taxes that are estimated to have been contributed in 1955 by the major classes of vehicles.

Registration fees

Registration fees are reported by all States by major type of vehicle. The only estimates in this instance are the fees attributable to camp and other light trailers and the distribution of truck fees between single-unit trucks and truck combinations.

Registration fees paid on passenger cars rose to $666 million in 1955, $62 million or 10 percent above the 1954 figure. An increase of $35 million or 8 percent brought the registration fees paid on trucks and truck combinations to $466 million. Although the contribution of passenger cars to the increase in total registration fees was greater than that of trucks and combinations, the effect on the percentage of total fees represented by each of the two classes of vehicles was negligible.

Registration fees paid on buses dropped about $1 million to a total of $11.7 million, a 7-percent decrease. This was the only reversal of the generally upward trend, but its effect on total fees is comparatively small since buses constitute only a 1-percent share. In the continued decline in the registration fees paid on this type of vehicle, the effect of a recent trend toward the granting of tax relief to buses operated for hire can be observed. Three States—Kentucky, North Carolina, and Wisconsin—were the chief contributors to the decrease.

Kentucky's bus seat fees of $8, $15, and $25, the maximum applying to all seats over 24, were reduced by a 1954 law to new rates of $5 per seat for the first 31 and $8 for each seat over that number. Consequently, bus registration fees paid to the State in 1955 decreased by two-thirds. Also largely because of rate reductions, bus registration fees paid to the States of North Carolina and Wisconsin fell off about one-third in 1955. A reduction from 6 to 3 percent in the rate of North Carolina's gross receipts tax on for-hire carriers, which is collected as part of the registration fee, took effect July 1, 1955. In Wisconsin a nominal fee of $1.00 per vehicle on certain city buses replaced a graduated weight tax having rates ranging from $20 for less than 3 tons to $60 for 5 tons plus $25 per ton over 5.

The effect of the reduced bus fees becomes more apparent when they are expressed in terms of averages per vehicle. The number of buses registered went up slightly in 1955, but the fees were $7 less per vehicle than they had been in 1954, falling from $90 to $83. The percentage decrease was about the same per vehicle as in total amount.

Registration fees on passenger cars and on trucks and truck combinations were affected by some upward revision of fee schedules and by a significant growth in the number of vehicles registered. The increase of 10 and 8 percent, respectively, in total fees attributable to these vehicle types represented a 3-percent average increase per vehicle in each case. Passenger-car fees averaged $13 per vehicle, almost the same as in 1954, and fees on trucks and truck combinations averaged $47. $1 more than the year before.

Miscellaneous taxes and fees

The other classes of highway-user taxes shown in table 4 also produced substantially higher returns in 1955 than in 1954. In 1955 motor-carrier taxes amounted to $91 million, 10 percent above the 1954 figure. There was a 30-percent increase in operator's and chauffeur's license fees, which reached $87 million in 1955, but the additional revenue from this source did not altogether represent a genuine increase. Much of it was the result of fluctuation produced by State requirements for periodic renewal. Miscellaneous fees rose 21 percent to $124 million. The $70 million of this amount which came from certificate-of-title fees and special titling taxes reflected the record volume of new car sales in 1955.

Miscellaneous highway-user taxes and fees have been allocated on the assumption—borne out by the 1955 registration fees—that in spite of changes in amount, a significant shift in the proportionate contributions of the principal vehicle types has not occurred. Rate changes, which might produce such a

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[3] *Traffic and travel trends, 1954*, by Thomas B. Dimmick, PUBLIC ROADS, vol. 28, No. 11, December 1955.

Table 5.—Estimate of average State highway-user taxes paid in 1955 per vehicle, per vehicle-mile, and per ton-mile

Vehicle type	Motor-vehicles registered [1]		Vehicle-miles traveled [1]		Highway-user taxes paid		Average rate of payment		
	Number	Distribution	Amount	Distribution	Amount [2]	Distribution	Per vehicle	Per vehicle-mile	Per ton-mile [3]
	Thousands	*Percent*	*Millions*	*Percent*	*1,000 dollars*	*Percent*	*Dollars*	*Cents*	*Cents*
Passenger cars	51, 989	83. 83	485, 761	81. 46	2, 621, 668	65. 82	50	0. 54	0. 27
Buses	142	. 22	3, 598	. 60	66, 709	1. 68	470	1. 85	. 20
Single-unit trucks	9, 345	15. 07	85, 731	14. 38	815, 085	20. 46	87	. 95	. 26
Combinations	544	. 88	21, 231	3. 56	479, 430	12. 04	881	2. 25	. 12
All trucks and combinations	9, 889	15. 95	106, 962	17. 94	1, 294, 515	32. 50	131	1. 21	. 18
All vehicles	62, 020	100. 00	596, 321	100. 00	3, 982, 892	100. 00	64	. 67	. 23

[1] Private and commercial vehicles. Publicly owned vehicles other than transit buses are omitted.

[2] Excludes fines and penalties amounting to $17,860,000, tax payments of $16,897,000 assigned to light trailers, and $8,008,000 assigned to motorcycles.

[3] Per ton-mile of average operating gross weight.

shift, have not been an important factor in determining 1955 yields except in the case of motor-fuel taxes.

Probably the most important change in rates of user taxes other than registration fees and motor-fuel taxes was made in Colorado, where two successive revisions of the mileage-tax law had the net effect of adding a tax of 0.8 mill per ton-mile on the empty weight of the vehicle to the original 2-mill rate per ton-mile on the cargo alone. There was a considerable increase in returns from this tax, but it is not clear how much of it was brought about by the rate change. It is reported that the larger yield has been officially ascribed to improved collection procedures following the inauguration of a port-of-entry system on July 1, 1955.[4]

Motor-fuel taxes

Between March 1 and September 13, 1955, higher motor-fuel tax rates went into effect on gasoline in 14 States and on special fuels in 15, the increased rate in one State being on special fuels only. In two additional jurisdictions, 1955 was the first full year of collections at higher rates introduced in 1954, making a total of 17 jurisdictions where 1955 fuel-tax revenues were influenced by rate changes. In all but two of these States, revenues were from 11 to 35 percent higher than they were in 1954. The rise in Pennsylvania's fuel-tax revenue was 2 percent, but here the rate increase, which became effective near the end of the year, was not fully reflected in collections reported for 1955. In Kansas where the rate increase was confined to special fuels, fuel-tax revenues increased 5 percent.

Fuel taxes produced revenues from 2 to 11 percent above the 1954 levels in the 31 States which had no rate change. In only one State did they show a decrease. This was South Dakota, where deductions—chiefly refunds—rose nearly 14 percent, more than offsetting the 3-percent increase in gross collections. Although the yield of motor-fuel taxes generally rose in response to extensive rate increases as well as to an increase in the number of vehicles, the distribution of the number of gallons consumed by type of vehicle remained steady.

Registration, travel, and taxes paid

The numbers of vehicles registered, the distances they traveled, and the highway-user taxes paid on them in 1955 are compared in table 5, which corresponds to table 3. An increase of 10.6 percent in user taxes between 1954 and 1955 was accompanied by a 7.2-percent increase in vehicle registrations and an estimated 7.6-percent increase in travel. Registrations and travel for 1955 show similar rates of increase for the principal vehicle types, passenger cars in the lead, followed by trucks and truck combinations and, finally, buses which exhibited very little change from 1954. The percentages of total registrations and total travel accounted for by passenger cars increased, though slightly, at the expense of trucks and buses. Taxes paid on the various types of vehicles showed a greater rate of increase than did numbers of vehicles or the vehicle-miles traveled. Thus, average rates of tax payments on the principal types of vehicles showed increases all along the line. Although the registration fees contributed by buses decreased, as shown in table 4, increased revenues from other user taxes, principally those on motor fuel, more than made up for the loss.

Payments on passenger cars increased from $48 to $50 per vehicle, from 0.52 to 0.54 cent per vehicle-mile, and from 0.26 to 0.27 per ton-mile. The increase in the average tax payment on single-unit trucks was from $84 to $87 per vehicle, from 0.92 to 0.95 cent per vehicle-mile, and from 0.24 to 0.26 per ton-mile. On combinations the average payment rose from $850 to $881 per vehicle, from 2.20 to 2.25 cents per vehicle-mile, but the payment per ton-mile remained 0.12 cent. Average tax payments on buses rose from $462 to $470 per vehicle, from 1.83 to 1.85 cents per vehicle-mile, and from 0.17 to 0.20 cent per ton-mile.

Vehicle Characteristics Affecting User-Tax Payments

Some discussion of the data upon which this study is based may be useful in evaluating or applying the findings. One of the first considerations was the classification of vehicles. This was also one of the principal problems encountered in making the estimates for 1952 reported in the earlier article. By the use of such information as was then available, a cross classification of trucks and truck combinations was developed in which these vehicles were classified on the basis of both simulated registered gross weight and the visual classification commonly used in traffic classification studies. Since the information needed to improve upon this analysis was not available, the percentage relations developed in 1952 were applied to the 1954 registrations to arrive at the distributions used in this article.

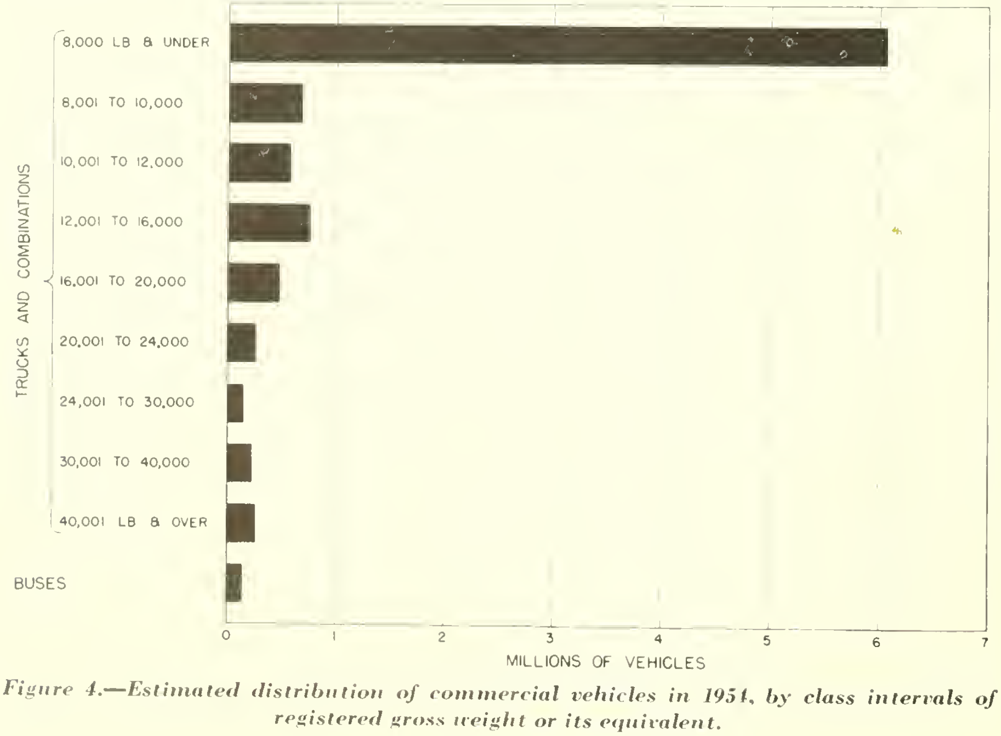

Figure 4.—Estimated distribution of commercial vehicles in 1954, by class intervals of registered gross weight or its equivalent.

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[4] *Automotive News*, Vol. 31, No. 3592, March 4, 1957, p. 23.

Table 6.—Estimated distribution of trucks and combinations in 1954, by visual class and registered gross weight or equivalent

Registered gross weight	Single-unit trucks						Vehicle combinations				Total	
	2-axle, 4-tire		2-axle, 6-tire		3-axle		Tractor-semitrailer		Truck-trailer			
	Number	Distribution	Number	Distribution	Number	Distribution	Number	Distribution	Number	Distribution	Number	Distribution
Pounds	*Thousands*	*Percent*	*Thousands*	*Percent*	*Thousands*	*Percent*	*Thousands*	*Percent*	*Thousands*	*Percent*	*Thousands*	*Percent*
6,000 and under	4,970	52.800	182	1.940	-----	-----	-----	-----	-----		5,152	54.740
6,001-8,000	583	6.200	326	3.460	-----	-----	-----	-----		-----	909	9.660
8,001-10,000	235	2.500	452	4.800	-----	-----	-----	-----	-----	-----	687	7.300
10,001-12,000	94	1.000	471	5.000	-----	-----	-----	-----		-----	565	6.000
12,001-16,000	-----	-----	700	7.440	19	0.200	31	0.330	3	0.030	753	8.000
16,001-20,000	-----	-----	411	4.360	28	.300	31	.330	1	.010	471	5.000
20,001-24,000	-----	-----	151	1.615	19	.200	78	.825	6	.060	254	2.700
24,001-30,000	-----	-----	65	.685	28	.300	39	.415	9	.100	141	1.500
30,001-40,000	-----	-----	66	.700	69	.735	87	.925	4	.040	226	2.400
Over 40,000	-----	-----	-----	-----	25	.265	205	2.175	24	.260	254	2.700
Total	5,882	62.500	2,824	30.000	188	2.000	471	5.000	47	.500	9,412	100.000

Table 7.—Estimated travel during 1954, by place of travel [1] and by type of vehicle

Vehicle type	Vehicle-miles of travel in—			Distribution of travel in—		
	Rural areas	Urban places	Total	Rural areas	Urban places	Total
	Millions	*Millions*	*Millions*	*Percent*	*Percent*	*Percent*
Passenger cars	246,300	204,305	450,605	77.64	83.85	80.34
School and miscellaneous buses	1,091	121	1,212	.34	.05	.22
Commercial buses:						
Intercity	1,190	16	1,206	.37	.01	.21
Transit	263	1,727	1,990	.09	.71	.36
All commercial buses	1,453	1,743	3,196	.46	.72	.57
All buses	2,544	1,864	4,408	.80	.77	.79
Single-unit trucks:						
2-axle, 4-tire	28,434	26,229	54,663	8.96	10.77	9.75
2-axle, 6-tire	21,477	6,032	27,509	6.77	2.48	4.90
3-axle	1,920	937	2,857	.61	.38	.51
All single-unit trucks	51,831	33,198	85,029	16.34	13.63	15.16
Vehicle combinations:						
Tractor-semitrailer	15,586	3,073	18,659	4.92	1.26	3.33
Truck-trailer	957	1,199	2,156	.30	.49	.38
All combinations	16,543	4,272	20,815	5.22	1.75	3.71
All trucks and combinations	68,374	37,470	105,844	21.56	15.38	18.87
All vehicles	317,218	243,639	560,857	100.00	100.00	100.00

[1] "Urban places" include all incorporated places and delimited urban compacts; the remainder is included in "rural areas".

Although numbers of vehicles are segregated in State registration records by major types, the further classifications required for allocating highway-user taxes in this analysis had to be estimated with the help of other sources of information. Since many States use some basis of registration other than gross vehicle weight, it was necessary to convert to gross-weight terms the data provided according to other weight or capacity groupings. This was done through the application of estimated conversion factors to the data for such States individually so as to arrive at approximate values for the registrations by gross-weight group that would have existed if all States required registration on this basis.

The classification of the total number of trucks and truck combinations, first according to gross weight and then according to the so-called visual class is illustrated in figures 4 and 5, and the two distributions are cross-classified in table 6. The greatest proportion of all trucks and truck combinations, as would be expected, falls in the lower weight groups. Of the total, an estimated 64.4 percent or 6,061,000 vehicles fall in the group 8,000 pounds or less, and 54.7 percent are in the group 6,000 pounds or below. The groups from 8,001 to 20,000 pounds account for an additional 26.3 percent or 2,476,000 vehicles, and only 9.3 percent of all trucks are estimated to be in the groups over 20,000 pounds gross weight. Of the heaviest vehicles, a 4.2-percent segment falls in the weight class of 20,001 to 30,000 pounds.

Determination of the taxes paid by various vehicles requires considerable knowledge of the mileages they travel. While registration fees generally do not vary with the amount of travel, motor-carrier taxes do to a great extent; and fuel taxes paid vary directly with travel. Travel data are computed primarily from observation, and the visual classification of vehicles shown in table 6 (reading across) and in figure 5 is that ordinarily used in recording and publishing traffic-volume information.

Since the number of truck registrations decreases with increasing gross weight, it is to be expected that the lighter types of single-unit trucks should appear in greatest numbers on the highways. On the other hand, since heavier vehicles tend to travel greater annual mileages, their frequency of occurrence in the traffic stream is out of proportion to their numbers. For this reason the evidence of traffic counts can only be used indirectly in estimating the numbers of vehicles in each visual class. According to table 6, 94.5 percent of all freight carrying vehicles are single-unit trucks; and, of these, 62.5 percent or 5,882,000 vehicles are of the 2-axle, 4-tire class. Another 30 percent or 2,824,000 vehicles have 2 axles and 6 tires. Only 188,000 or 2 percent are 3-axle, single-unit vehicles.

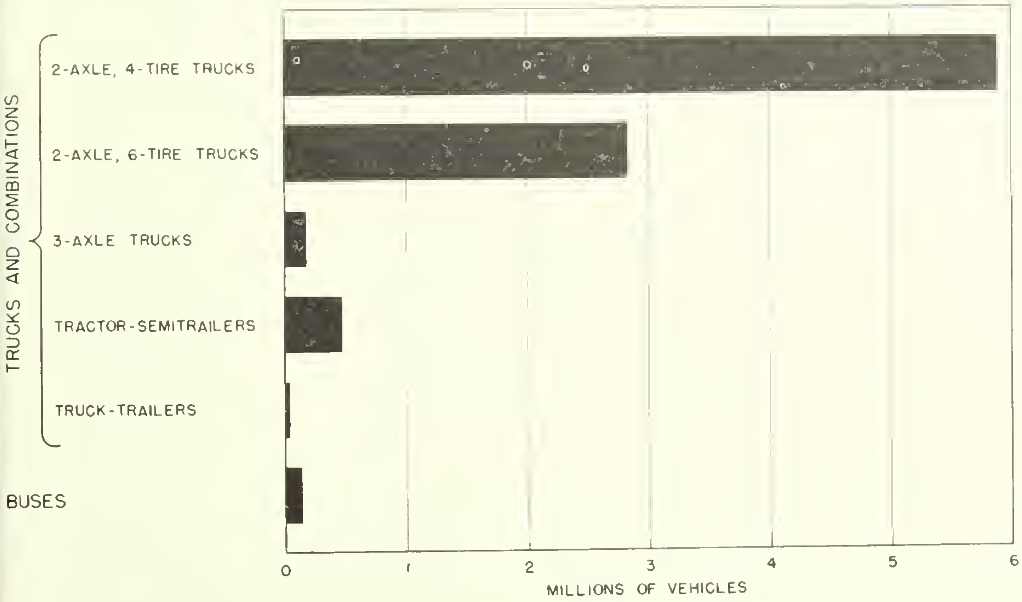

Figure 5.—Estimated distribution of commercial vehicles in 1954, by vehicle types.

Combinations number 518,000 or 5.5 percent of the total, of which one-half of 1 percent are truck-trailer combinations.

As stated at the outset, the classification of buses is by type of operation and ownership rather than by weight. This was made necessary by the wide differences in the tax liability of vehicles engaged in intercity commercial operation, in transit operation, and in nonrevenue operation.

Motor-Vehicle Travel

In order to calculate the fuel-tax payments of individual types of vehicles, the estimated travel during 1954, rural and urban, already available for major types of vehicles,[5] was further subdivided as shown in table 7. Total motor-vehicle travel on all roads and streets during 1954 was estimated to be 561 billion vehicle-miles. Passenger-car travel accounted for 80 percent of this amount or 451 billion vehicle-miles. All trucks and truck combinations contributed 19 percent of the travel or 106 billion vehicle-miles, 15 percent by single-unit trucks and the remaining 4 percent by combinations. Buses traveled only 1 percent of the total or 4 billion vehicle-miles.

Since the purposes of this analysis require that the calculation of fuel consumption and fuel-tax payments be limited to the classes of private and commercial vehicles shown in table 6 and figure 5, it was necessary to eliminate the travel of publicly owned vehicles from the estimated travel of all vehicles. Estimates for Federal civilian vehicles were determined from statistics compiled by the U. S. Bureau of the Budget, and those for vehicles of other governmental agencies were developed from reports of State highway departments.

Table 8.—Estimated travel during 1954, by type and ownership of vehicle

Vehicle type	Vehicle-miles of travel by—			Distribution of travel by—		
	Government-owned vehicles	Private and commercial vehicles	Total	Government-owned vehicles	Private and commercial vehicles	Total
	Millions	*Millions*	*Millions*	*Percent*	*Percent*	*Percent*
Passenger cars	1, 692	448, 913	450, 605	24. 64	81.03	80. 34
School and miscellaneous buses	869	343	1, 212	12. 66	. 06	. 22
Commercial buses:						
Intercity	--------	1, 206	1, 206	--------	. 22	. 21
Transit	--------	1, 990	1, 990	--------	. 36	. 36
All commercial buses	--------	3, 196	3, 196	--------	. 58	. 57
All buses	869	3, 539	4, 408	12. 66	. 64	. 79
Single-unit trucks:						
2-axle, 4 tire	1, 921	52, 742	54, 663	27. 98	9. 52	9. 75
2-axle, 6-tire	1, 511	25, 998	27, 509	22. 00	4. 69	4. 90
3-axle	87	2, 770	2, 857	1. 27	. 50	. 51
All single-unit trucks	3, 519	81, 510	85, 029	51. 25	14. 71	15. 16
Vehicle combinations:						
Tractor-semitrailer	730	17, 929	18, 659	10. 63	3. 24	3. 33
Truck-trailer	56	2, 100	2, 156	. 82	. 38	. 38
All combinations	786	20, 029	20, 815	11. 45	3. 62	3. 71
All trucks and combinations	4, 305	101, 539	105, 844	62. 70	18. 33	18. 87
All vehicles	6, 866	553, 991	560, 857	100. 00	100. 00	100. 00

Total travel shown in table 7 is classified by vehicle ownership in table 8, and that portion attributable to private and commercial vehicles only is displayed by visual class in figure 6. Government-owned nonmilitary vehicles

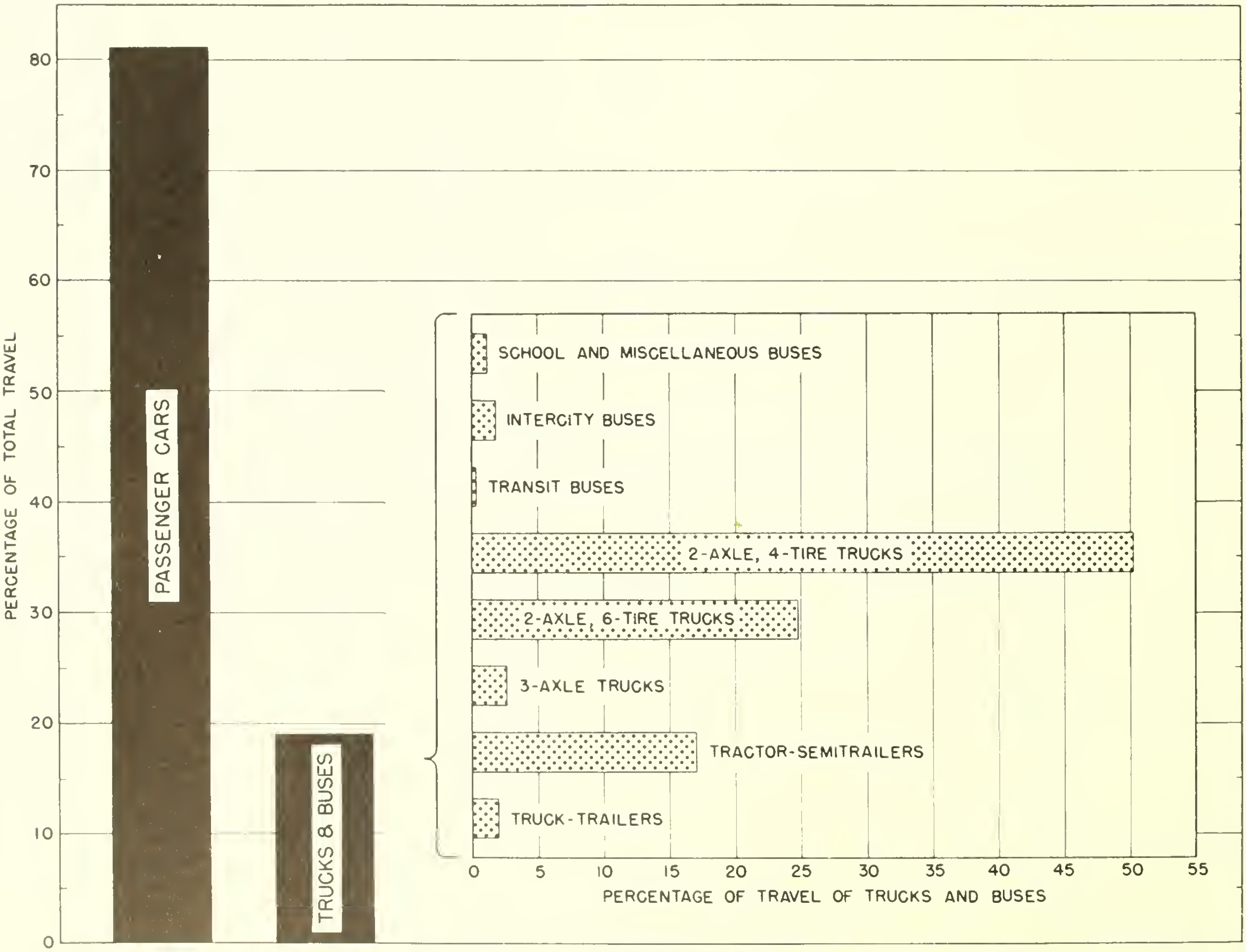

Figure 6.—Percentage distribution of travel during 1954 by passenger cars and commercial vehicles.

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[5] See footnote 3, p. 283.

Table 9.—Estimated operating characteristics of various types of motor vehicles

Vehicle type	Average operating gross weight		Distribution of travel by type of fuel used			Rate of fuel consumption, by type of fuel used						
						Per mile			Per ton-mile			
									Gasoline		Diesel	
	Amount	Index	Gasoline	Diesel	Other	Gasoline	Diesel	Other	Amount	Index	Amount	Index
	Pounds		*Percent*	*Percent*	*Percent*	*Gallons*	*Gallons*	*Gallons*	*Gallons*		*Gallons*	
Passenger cars	4,025	1.00	100.0	(1)	(1)	0.067	--------	------	0.034	1.000	------	------
School and miscellaneous buses	11,600	2.88	100.0	(1)	(1)	.125	--------	------	.022	.647	------	------
Commercial buses:												
Intercity	23,000	5.71	38.9	61.1	(1)	.185	0.135	--------	.016	.471	0.012	0.353
Transit	23,000	5.71	60.2	35.4	4.4	.307	.228	0.364	.027	.794	.020	.588
Single-unit trucks:												
2-axle, 4-tire	4,764	1.84	100.0	(1)	(1)	.074	--------	--------	.031	.912	------	------
2-axle, 6-tire	11,290	2.96	100.0	(1)	(1)	.127	--------	--------	.021	.618	------	------
3-axle	24,102	5.99	100.0	(1)	(1)	.192	--------	--------	.016	.471	------	------
Vehicle combinations:												
Tractor-semitrailer	35,690	8.87	79.9	17.9	2.2	.241	.168	.268	.013	.382	.009	.265
Truck-trailer	46,799	11.63	79.9	17.9	2.2	.282	.197	.313	.012	.353	.008	.235

[1] Percentage negligible.

other than publicly owned transit vehicles were estimated to have traveled 7 billion vehicle-miles in 1954, leaving 554 billion vehicle-miles of travel as the contribution of private and commercial vehicles. Of this amount passenger cars accounted for 449 billion; trucks and truck combinations, 101.5 billion; and buses, 3.5 billion. The percentage distribution of this travel by vehicle groups is little changed by subtraction of the travel of publicly owned vehicles.

Vehicle Operating Characteristics

Estimating the fuel consumption and fuel-tax payments of the classes of vehicles used in this study made it necessary to determine certain of their operating characteristics, such as average gross weights, percentages of vehicles using fuel other than gasoline, and rates of fuel consumption. The results of these calculations are shown in table 9.

Use of fuels other than gasoline

While special fuels such as diesel and propane and other liquefied petroleum gases still constitute a relatively small part of the total fuel consumed on the highways—1 billion gallons out of 44 billion in 1954[6]—the use of these fuels is almost entirely confined to the larger commercial vehicles and comprises a substantial portion of their fuel consumption. The estimates in table 9 show a percentage relation of 80–20 between the miles of travel accounted for by truck combinations using gasoline and those using special fuels, with diesel fuel accounting for most of the 20 percent.

On the other hand, information reported by the commercial bus industry supports the estimate presented in table 9 that special fuels account for 61 percent of the miles traveled by intercity buses. Fuel usage reported by cities having publicly owned transit systems shows a similar relation, special fuels accounting for 58 percent of the travel of vehicles operated by these systems. The data reported by cities also show the greatest use of propane, which accounted for 13 percent of the travel of publicly owned transit buses. The use of special fuels by privately owned transit vehicles is considerably less than this, and averaging the two reduces to 39.8 percent the proportion of all intracity-bus travel accounted for by special fuels.

Estimates of fuel-consumption rates

The rates of gasoline consumption for passenger cars, trucks and truck combinations, and noncommercial buses were obtained from the equation developed for the 1952 study to indicate approximate gasoline-consumption rates for gross vehicle weights up to at least 72,000 pounds under average operating conditions. This equation represents a composite of values for numerous gross weight groups obtained from each of the several previous determinations by other investigators.

All of the gasoline rates shown in table 9 were developed by applying the derived equation to the average operating gross weight shown, except in the case of commercial buses. Fuel-consumption rates for diesel and other fuels were based on information obtained from representatives of the trucking industry and the various segments of the bus industry.

Rates of diesel-fuel consumption by truck combinations were determined on the basis of the assumption, arrived at after consultation with representatives of the trucking industry, that average fuel consumption of diesel ve-

Table 10.—Fuel consumption and tax payments in 1954, by major types of private and commercial vehicles

Vehicle type	Total miles traveled	Gasoline-powered vehicles		Diesel-powered vehicles		Vehicles powered by other fuels		Total fuel consumed		Total tax payments
		Mileage	Fuel consumed	Mileage	Fuel consumed	Mileage	Fuel consumed	Gallons	Distribution	
	Millions	*Millions*	*Million gallons*	*Millions*	*Million gallons*	*Millions*	*Million gallons*	*Millions*	*Percent*	*Million dollars*
Passenger cars	448,913	448,913	30,288	--------	--------	--------	--------	30,288	69.503	1,602.6
School and miscellaneous buses	343	343	43	--------	--------	--------	--------	43	.099	2.3
Commercial buses:										
Intercity	1,206	469	87	737	100	--------	--------	187	.429	9.9
Transit	1,990	1,196	367	706	161	88	32	560	1.285	29.6
All commercial buses	3,196	1,665	454	1,443	261	88	32	747	1.714	39.5
All buses	3,539	2,008	497	1,443	261	88	32	790	1.813	41.8
Single-unit trucks:										
2-axle, 4-tire	52,742	52,742	3,913	--------	--------	--------	--------	3,913	8.979	207.0
2-axle, 6-tire	25,998	25,998	3,303	--------	--------	--------	--------	3,303	7.580	174.8
3-axle	2,770	2,770	531	--------	--------	--------	--------	531	1.218	28.1
All single-unit trucks	81,510	81,510	7,747	--------	--------	--------	--------	7,747	17.777	409.9
Vehicle combinations:										
Tractor-semitrailer	17,929	14,325	3,450	3,209	541	395	106	4,097	9.402	216.8
Truck-trailer	2,100	1,678	473	376	74	46	14	561	1.287	29.7
All combinations	20,029	16,003	3,923	3,585	615	441	120	4,658	10.689	246.5
All trucks and combinations	101,539	97,513	11,670	3,585	615	441	120	12,405	28.466	656.4
All vehicles	553,991	548,434	42,455	5,028	876	529	152	43,483	99.782	2,300.8
Fuel consumed by motorcycles, etc	--------	--------	--------	--------	--------	--------	--------	95	.218	5.0
Total fuel consumed and tax payments	--------	--------	--------	--------	--------	--------	--------	43,578	100.000	2,305.8

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[6] *Highway Statistics, 1954,* published by the Bureau of Public Roads in 1955, tables G–21 and G–25.

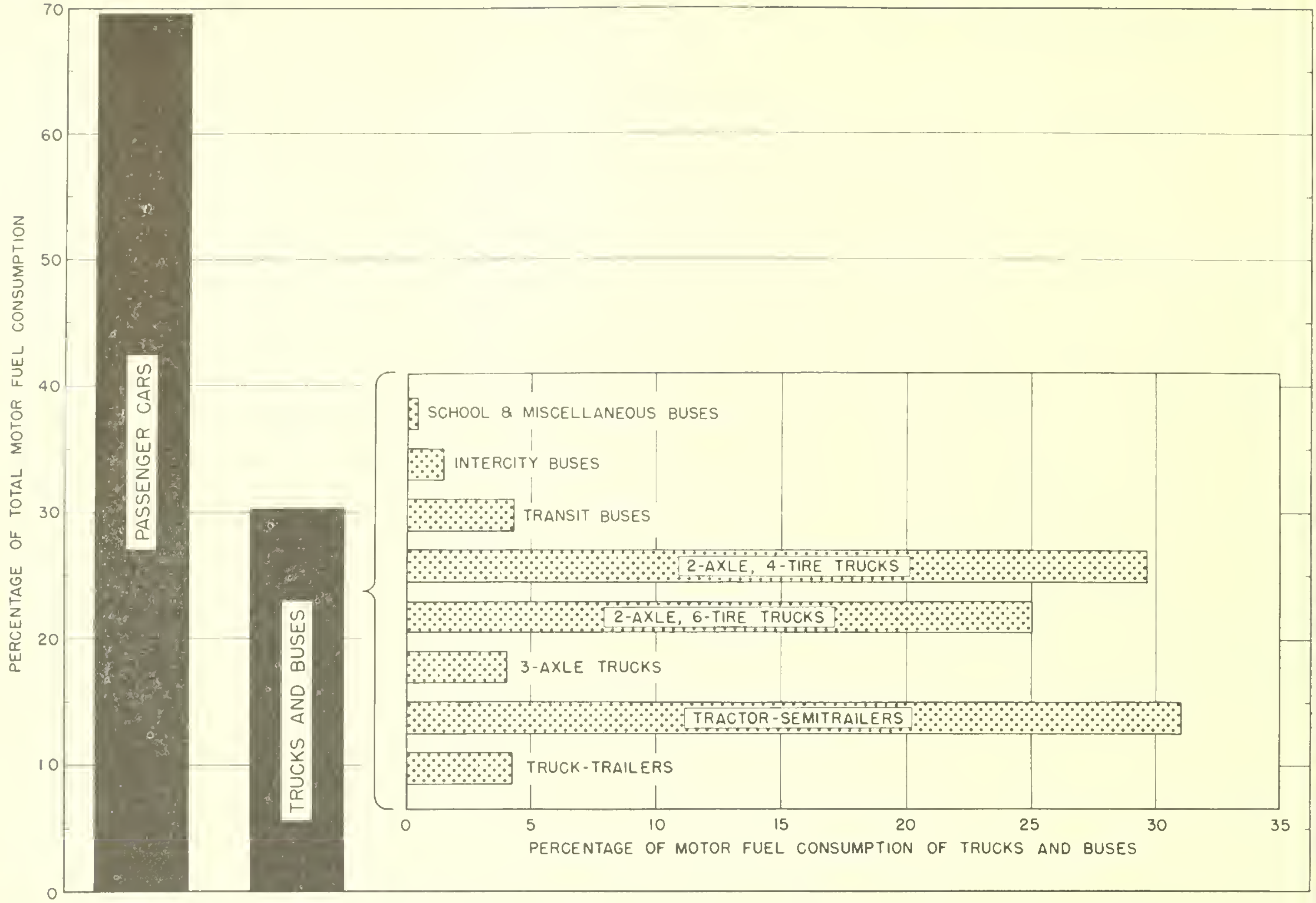

Figure 7.—Percentage distribution of motor-fuel consumption during 1954 by passenger cars and commercial vehicles.

hicles with operating gross weights above 20,000 pounds will be about 30 percent less than that of gasoline-powered vehicles of equal weight. The rate of consumption of special fuels other than diesel, when used in truck combinations, was taken as one-ninth greater than that of gasoline.

Fuel consumption and fuel-tax payments

The calculated fuel consumption for the visual classes of vehicles used in this study is shown in table 10, and the percentage distribution of the total is shown in figure 7. The fuel consumption was obtained using the total mileages shown in table 10 and the rates given in table 9.

The 43,483 million gallons of fuel of all kinds consumed in 1954 are the total reported by State agencies less 95 million gallons or 0.218 percent estimated to have been consumed by motorcycles, motor-scooters, and like vehicles.

Total fuel-tax payments in 1954 amounted to $2.31 billion. The fuel consumption represented by these tax payments is not quite the same as that reported above because the tax figure represents collections of motor-fuel taxes during the year, while the fuel-consumption figure represents fuel actually purchased during the year. In order to minimize any resulting imbalance between the two, a percentage distribution of fuel consumed was applied to the total tax payments instead of calculating the payments directly from the gallonage distribution.

Summary

Perhaps the results of this analysis are best summarized in the "Regrouping of vehicle types" found at the bottom of table 3. Here it is shown that in 1954 passenger cars and light trucks comprised about 94 percent of the registered motor vehicles and accounted for approximately 91 percent of the vehicle-miles of travel and 74 percent of the State highway-user taxes paid. In contrast, medium and heavy trucks and truck combinations as a group accounted for 6 percent of the vehicles registered, about 9 percent of the travel, and 24 percent of the user taxes.

Average tax payments per vehicle of $49 for the light vehicles and $244 for the heavier ones show the tendency of these payments to increase sharply with size and weight. Less difference is shown between the average payments of the two groups per vehicle-mile, 0.53 cent for passenger cars and light trucks and 1.76 cents for medium and heavy trucks and truck combinations, since the effect of the high annual mileages traveled by the heavy vehicles has been factored out. On a ton-mile basis the distinction is reversed, the light vehicles averaging 0.26 cent compared with 0.15 cent per ton-mile for the medium and heavy vehicles.

Because of the importance of the findings of studies of this kind, it is essential to employ analytical methods as precise as it is possible to make them. Those developed for the 1952 study, which were adopted with minor modifications for this one, are believed to be as satisfactory for a nationwide analysis as the data now available will permit. Yet, while the estimates presented here are regarded as sufficiently valid to be useful, the gaps in the basic data are still considerable.

More reliable estimates will be possible in the future as further information becomes available and as these additions to the basic data contribute to improved analytical techniques. Broadened State traffic-counting and weighing operations and additional motor-vehicle-use studies are likely sources of new information. The measurement of highway use is one of the principal requirements of the study of highway cost allocation called for by section 210 of the Federal Highway Revenue Act of 1956. It is hoped that auxiliary data produced for this study, such as estimates of vehicle registrations and traffic volumes by type and weight class of vehicle, will make possible a more accurate corresponding classification of tax payments.

U. S. GOVERNMENT PRINTING OFFICE: 1958

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Bibliography of Highway Planning Reports (1950). 30 cents.
Braking Performance of Motor Vehicles (1954). Out of print.
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First Progress Report of the Highway Cost Allocation Study, House Document No. 106 (1957). 35 cents.
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Highway Capacity Manual (1950). $1.00.
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Highway Practice in the United States of America (1949). 75 cents.
Highway Statistics (annual):
- 1945 (out of print).
- 1946, 50 cents.
- 1947 (out of print).
- 1948, 65 cents.
- 1949, 55 cents.
- 1950 (out of print).
- 1951, 60 cents.
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- 1953, $1.00.
- 1954, 75 cents.
- 1955, $1.00.

Highway Statistics, Summary to 1955. $1.00.
Highways in the United States, *nontechnical* (1954). 20 cents.
Highways of History (1939). 25 cents.
Identification of Rock Types (reprint from PUBLIC ROADS, June 1950). 15 cents.
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Legal Aspects of Controlling Highway Access (1945). 15 cents.
Local Rural Road Problem (1950). 20 cents.
Manual on Uniform Traffic Control Devices for Streets and Highways (1948) (*including 1954 revisions supplement*). $1.25.
  Revisions to the Manual on Uniform Traffic Control Devices for Streets and Highways (1954). *Separate,* 15 cents.

PUBLICATIONS (Continued)

Mathematical Theory of Vibration in Suspension Bridges (1950). $1.25.
Needs of the Highways Systems, 1955–84, House Document No. 120 (1955). 15 cents.
Opportunities in the Bureau of Public Roads for Young Engineers (1955). Out of print.
Parking Guide for Cities (1956). 55 cents.
Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft (1943). $2.00.
Progress and Feasibility of Toll Roads and Their Relation to the Federal-Aid Program, House Document No. 139 (1955). 15 cents.
Public Control of Highway Access and Roadside Development (1947). 35 cents.
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Results of Physical Tests of Road-Building Aggregate (1953). $1.00.
Roadside Improvement, No. 191MP (1934). 10 cents.
Selected Bibliography on Highway Finance (1951). 60 cents.
Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways, 1956: a reference guide outline. 55 cents.
Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, FP–57 (1957). $2.00.
Standard Plans for Highway Bridge Superstructures (1956). $1.75.
Taxation of Motor Vehicles in 1932. 35 cents.
Tire Wear and Tire Failures on Various Road Surfaces (1943). 10 cents.
Transition Curves for Highways (1940). $1.75.

MAPS

State Transportation Map series (available for 39 States). Uniform sheets 26 by 36 inches, scale 1 inch equals 4 miles. Shows in colors Federal-aid and State highways with surface types, principal connecting roads, railroads, airports, waterways, National and State forests, parks, and other reservations. Prices and number of sheets for each State vary—see Superintendent of Documents price list 53.
United States System of Numbered Highways. 28 by 42 inches, scale 1 inch equals 78 miles. 20 cents.

Single copies of the following publications are available to highway engineers and administrators for official use, and may be obtained by those so qualified upon request addressed to the Bureau of Public Roads. They are not sold by the Superintendent of Documents.

Bibliography on Automobile Parking in the United States (1946).
Bibliography on Highway Lighting (1937).
Bibliography on Highway Safety (1938).
Bibliography on Land Acquisition for Public Roads (1947).
Bibliography on Roadside Control (1949).
Express Highways in the United States: a Bibliography (1945).
Indexes to PUBLIC ROADS, volumes 17–19 and 23.
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U. S. DEPARTMENT OF COMMERCE - BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF DECEMBER 31, 1957

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			CONTRACTS ADVERTISED, CONSTRUCTION NOT STARTED			PROJECTS UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$33,714	$75,766	$50,776	499.1	$20,906	$14,363	164.6	$72,633	$45,862	816.5	$169,305	$111,001	1,480.2
Arizona	15,430	30,371	26,732	168.5	9,843	8,468	64.3	28,367	24,172	184.8	68,581	59,372	417.6
Arkansas	26,326	49,111	38,480	552.7	16,231	11,181	131.1	45,067	31,986	341.4	110,409	81,647	1,025.2
California	63,338	50,724	38,046	197.5	49,003	42,438	61.4	565,349	243,384	340.9	665,076	323,868	599.8
Colorado	40,592	19,243	14,364	122.0	1,999	1,324	21.5	57,210	41,977	303.3	78,452	57,665	446.8
Connecticut	52,140	1,766	1,048	.9	5,703	3,858	1.9	40,293	27,610	36.0	47,762	32,516	38.8
Delaware	30,807	3,963	1,998	9.9	766	395	.7	14,279	8,208	73.6	19,008	10,601	84.2
Florida	11,428	75,734	58,547	376.3	15,885	11,010	60.8	58,214	39,320	265.4	149,833	108,877	702.5
Georgia	32,363	114,615	85,324	745.1	19,405	12,724	110.0	96,915	54,515	905.1	230,935	152,563	1,760.2
Idaho	41,485	15,877	12,639	94.9	2,491	1,864	38.3	18,275	12,988	135.3	36,643	27,491	268.5
Illinois	63,523	94,609	68,758	438.0	90,931	69,166	134.3	184,340	134,014	685.7	369,880	271,938	1,258.0
Indiana	133,810	35,078	20,018	217.2	15,471	8,081	217.5	55,119	34,491	275.9	105,668	62,590	710.6
Iowa	33,469	55,650	41,412	690.9	28,658	22,327	203.5	55,681	37,851	1,139.4	139,989	101,590	2,033.8
Kansas	30,194	34,144	26,474	780.1	23,764	18,221	183.8	54,096	37,816	1,097.0	112,004	82,511	2,060.9
Kentucky	55,663	38,037	26,592	122.3	6,432	4,634	24.5	64,946	45,151	232.1	109,415	76,377	378.9
Louisiana	35,683	64,705	45,709	334.2	28,063	19,267	90.6	56,650	31,513	342.1	149,418	96,489	766.9
Maine	33,645	9,984	7,374	28.0	3,983	3,090	8.0	17,172	10,335	91.5	31,139	20,799	127.5
Maryland	13,074	24,837	14,060	126.1	26,927	18,376	44.9	72,452	51,904	165.9	124,216	84,340	336.9
Massachusetts	39,791	53,259	40,353	30.3	50,135	34,004	25.8	85,356	52,944	58.5	188,750	127,301	114.6
Michigan	76,330	87,475	67,985	498.9	52,894	32,538	147.6	85,746	63,185	302.5	226,115	163,708	949.0
Minnesota	60,870	9,229	7,856	58.3	6,886	4,404	62.7	104,546	79,203	919.6	120,661	91,463	1,040.6
Mississippi	19,533	50,005	36,203	657.3	25,874	21,212	119.9	58,737	39,325	790.4	134,616	96,740	1,567.6
Missouri	36,992	61,980	43,335	1,019.9	10,231	6,885	41.8	108,434	72,762	1,055.9	180,645	122,982	2,117.6
Montana	64,756	9,717	7,204	163.3	6,319	4,381	53.4	44,739	33,320	331.0	60,775	44,905	547.7
Nebraska	61,754	10,313	6,022	202.5	7,931	4,280	104.4	58,403	39,574	1,028.6	76,647	49,876	1,335.5
Nevada	31,360	11,031	10,272	67.1	3,041	2,775	4.5	24,729	22,695	105.4	38,801	35,742	177.0
New Hampshire	18,870	8,153	5,518	26.7	2,555	2,144	2.8	22,455	15,330	54.3	33,163	22,992	83.8
New Jersey	96,271	9,168	5,087	59.4	14,985	10,729	11.2	69,558	49,875	46.1	93,711	65,691	116.7
New Mexico	17,401	13,015	10,145	61.8	5,016	3,750	51.5	44,708	36,923	303.2	62,739	50,818	416.5
New York	130,441	49,755	35,441	102.1	73,327	53,918	46.7	506,650	321,496	390.8	629,732	410,855	539.6
North Carolina	93,500	32,763	20,487	313.2	11,035	9,084	35.4	81,997	51,728	738.9	125,795	81,299	1,087.5
North Dakota	18,759	33,475	25,169	1,007.6	11,984	10,086	110.3	24,088	16,515	969.9	69,547	51,770	2,087.8
Ohio	24,611	60,423	33,171	158.4	114,658	94,854	115.3	199,574	134,395	293.1	374,655	262,420	566.8
Oklahoma	28,536	61,051	42,282	580.4	23,752	19,923	76.3	63,228	40,380	567.0	148,031	102,585	1,223.7
Oregon	35,824	11,113	8,836	46.7	8,587	6,574	34.4	37,734	29,592	183.2	57,434	45,002	264.3
Pennsylvania	149,835	81,601	50,216	170.6	31,858	22,089	60.5	253,683	167,409	377.5	367,142	239,714	608.6
Rhode Island	12,099	10,885	8,075	8.8	3,320	2,848	1.7	32,096	21,257	21.1	46,301	32,180	31.6
South Carolina	21,010	67,044	51,174	644.6	7,081	4,776	49.4	42,030	26,308	642.9	116,155	82,258	1,336.9
South Dakota	25,268	45,196	36,517	610.2	1,165	751	18.1	19,792	14,512	402.1	66,153	51,780	1,030.4
Tennessee	44,356	64,142	50,113	393.4	26,758	18,411	68.0	86,259	52,759	472.4	177,159	121,283	933.8
Texas	161,064	49,419	36,891	592.4	66,941	50,795	189.1	189,578	130,667	1,742.7	305,938	218,353	2,524.2
Utah	10,328	32,125	29,022	200.6	7,790	6,654	45.2	21,565	18,174	84.6	61,480	53,850	330.4
Vermont	16,278	11,493	9,292	33.9				19,995	15,060	33.1	31,488	24,352	67.0
Virginia	22,871	48,857	38,330	130.2	34,276	27,429	56.4	69,896	48,798	239.8	153,029	114,557	426.4
Washington	49,987	22,168	14,095	147.4	3,630	3,075	13.9	55,284	41,602	220.9	81,082	58,772	382.2
West Virginia	42,265	43,056	32,598	67.5	4,951	2,477	22.5	38,366	21,176	69.8	86,373	56,251	159.8
Wisconsin	91,455	21,467	14,061	243.7	12,072	7,787	121.2	55,036	36,696	365.1	88,575	58,544	730.0
Wyoming	5,718	24,736	20,332	170.8	1,719	1,136	18.7	45,423	38,282	260.8	71,878	59,750	450.3
Hawaii	4,688	10,600	5,254	17.0	3	1		3,532	1,723	4.4	14,135	6,978	21.4
District of Columbia	27,531	6,257	4,757	2.8	4,173	2,936	1.3	20,701	16,514	4.4	31,131	24,207	8.5
Puerto Rico	11,459	12,420	5,929	31.7	59	18		18,196	8,409	51.0	30,675	14,356	82.7
Alaska	14,236	2,930	2,930	46.3	1,410	1,410	28.2	10,680	9,173	330.8	15,020	13,513	405.3
TOTAL	2,312,731	1,920,535	1,393,303	14,069.5	1,002,877	744,921	3,299.9	4,159,852	2,680,858	20,893.7	7,083,264	4,819,082	38,263.1

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